MECHANISTIC STRUCTURAL ANALYSIS AND DESIGN OF RECYCLED AGGREGATES IN ROAD CONSTRUCTION CASE STUDY: CITY OF SASKATOON

A Thesis Submitted to the College of

Graduate Studies and Research

in Partial Fulfillment of the Requirements

for the Degree of Master of Science

in the Department of Civil and Geological Engineering

University of Saskatchewan

Saskatoon

By

Duane Joel Guenther

PERMISSION TO USE

In presenting this thesis in partial fulfillment of the requirements for a Postgraduate degree from the University of Saskatchewan, the author has agreed that the Libraries of this University may make it freely available for inspection. The author has further agreed that permission for copying of this thesis in any manner, in whole or in part, for scholarly purposes may be granted by the professor or professors who supervised the thesis work or, in their absence, by the Head of the Department or the Dean of the College in which the thesis work was done. It is understood that any copying, publication, or use of this thesis or parts thereof for financial gain shall not be allowed without the author's written permission. It is also understood that due recognition shall be given to the author and to the University of Saskatchewan in any scholarly use which may be made of any material in this thesis.

Requests for permission to copy or to make other use of material in this thesis in whole or part should be addressed to:

Department of Civil & Geological Engineering

57 Campus Drive

University of Saskatchewan

Saskatoon, Saskatchewan, SK S7N 5A9

Canada

ABSTRACT

The current manner of constructing roads with virgin aggregates is unsustainable for many urban centers as natural sources for quality aggregates are being or have been depleted. As well, there is little understanding or scientific data published as to the impacts on roadway design and life cycle performance with poorer quality aggregate material. To improve future sustainability of roadway utility, there is a need for better understanding of the mechanistic behavior of road aggregates and their respective role in road structural performance in the field. As well, there is a need to find more sustainable sources of quality aggregates to construct roadways.

The goal of this research is to improve road utility sustainability through a better understanding of life cycle performance and incorporating field state mechanistic principles in the initial design of the roadway structure.

The primary objective of this research was to investigate the application of recycle rubble materials using a mechanistic materials characterization and structural design process for urban roadways within typical City of Saskatoon roads and field state conditions. Specific technical objectives of this research were to characterize various recycled aggregate materials with regards to their road structural behavior as a high quality base coarse, quantify the cost comparison between various design cross sections, and evaluate the structural behavior of these alternate aggregate sources in typical structural designs and Saskatoon field state conditions. To validate the field behavior of recycled aggregates, various test sections were constructed with different recycled and virgin aggregate structural systems. These test sections were evaluated using non-destructive structural assessment to determine their structural quality in the field.

This research studied the use of recycled portland cement concrete aggregates and recycled asphalt cement aggregates as structural granular layers of typical City of Saskatoon roadways. These materials were characterized using conventional and mechanistic lab characterization protocols. Field test sections were constructed to validate that recycled materials could be employed as quality replacements for virgin aggregates. Research was also conducted on how to incorporate mechanistic based materials testing and structural design into City of Saskatoon Design and Materials Selection Specifications and Processes. The resilient

modulus of the various road materials was also compared to relate to other mechanistic-empirical design methodologies.

The laboratory testing conducted in this research indicated that although conventional empirical testing shows recycled asphalt materials to be of lesser quality, when evaluated using mechanistic characterization protocols, recycled asphalt concrete material yielded superior structural behavior. To illustrate, the dynamic modulus of recycled asphalt concrete was 697 MPa under a fully reversed stress state and a frequency of 0.5 Hz compared to 264 MPa for a high quality conventional high fracture granular base under the same stress state and frequency. As well, the recycled asphalt material showed less moisture susceptibility than conventional granular aggregate.

This research showed recycled portland cement concrete aggregate materials showed good drainage and capillary break qualities when tested against the standard granular base materials. Although the well graded recycled asphalt cement and well graded recycled portland cement concrete were shown to have slightly higher moisture intake values, the increased moisture did not increase the swell and therefore indicates adequate frost resistance due to moisture.

This research showed conventional roadway design utilized by the City of Saskatoon does not have the means to evaluate recycled asphalt and portland cement aggregates from a materials selection and structural design perspective. Roadway designs using a mechanistic approach were able to accurately represent the field structural primary responses of test roadway structures considered in this study and were able to incorporated recycled aggregate in the design process. Designing roads using a mechanistic design process showed a significant improvement in roadway structural responses in designs using recycled aggregate material.

From an economic perspective, this research showed road cross sections utilizing recycle aggregate materials proved to be the least expensive option when evaluated by the initial capital cost and the projected life cycle costing. When comparing primary structural responses to construction cost, up to 20 percent of costs to construct a road can be saved, and a properly designed road structure using recycled aggregates will reduce the strains in the structures by up to 90 percent.

As well, using recycled aggregates to construct roadways will reduce the fuel consumption during construction by up to 20 percent due to a reduction in aggregate hauling distances.

In summary, when evaluated with a mechanistic road structural design method that accounts for the material characteristics of various aggregates, recycled asphalt rubble processed as a black base and recycled portland cement concrete as a stress dissipating drainage layer within the construction of new roadways is a more sustainable approach to designing and constructing structurally sound roads than the conventional methods.

Based on the findings of this research, proper stockpiling and recycling of asphalt and concrete rubble materials is recommended in the City of Saskatoon. In order to optimize and incorporate various recycled aggregate materials into road design it is recommended the City of Saskatoon employ mechanistic based road material characterization and structural design.

ACKNOWLEDGMENTS

This research would not have been possible without help and guidance from a number of individuals. Without the big push from my supervisor, Dr. Curtis Berthelot, I would not have even started this journey. His guidance and encouragement helped me stick with it and push through adversity. Thanks as well to my advisory committee, Dr. Peter Park, Dr. Gordon Sparks, and Dr. Gordon Putz, for their guidance and advice.

Thanks also to those from the City of Saskatoon Public Works Department who helped construct the test sections and joined me in taking the risk of constructing roads with unconventional materials. Your skills, experience and hard work helped build better roads.

As lab work and computer modelling are a major part of this research, a special thanks to Rielle Haichert for all the hours of lab work involved in developing the material characteristics and Roberto Soares for his finite element model and assistance in producing the model results.

Lastly, I would like to thank my wife, Elsa, and children, Jael, Micah and Isaac, for putting up with the vast hours that I needed to be away from them to complete this research. Their understanding and patience have been outstanding.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

List of Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
AC	Asphalt Cement
ADT	Annual Daily Traffic
ASTM	American Society for Testing and Materials
CBR	California Bearing Ratio
CH	Highly Plastic Clay
CIP	Cold in Place
COS	City of Saskatoon
CR	Crushed Rock
ESAL	Equivalent Single Axle Load
FEM	Finite Element Method
GB	Granular Base
GIS	Geographic Information System
GP	Poorly Graded
GW	Well Graded
HF	High Fines
HMA	Hot Mix Asphalt
HWD	Heavy Weight Deflectometer
NCHRP	National Cooperative Highway Research Program
OGBC	Open Graded Base Course
PCC	Portland Cement Concrete
PIARC	Permanent International Association of Road Congress
RAP	Reclaimed Asphalt Product
RaTT	Rapid Triaxial Tester
SB	Sub-base
SM-SC	Clayey Silty Sand
SMHI	Saskatchewan Ministries of Highways and Infrastructure
USCS	Unified Soil Classification System
VacSat	Vacuum Saturated
WYDOT	Wyoming Department of Transportation

CHAPTER 1 INTRODUCTION

1.1 Background

Roadway design and construction in the City of Saskatoon (COS) has predominantly employed the same engineering processes for the past 50 years (City of Saskatoon, 2008). The design for City of Saskatoon roadways has traditionally been based on the Saskatchewan Ministry of Highways and Infrastructure (SMHI) modified California Bearing Ratio (CBR) design nomographs with certain assumptions associated with primary rural highway design. The assumptions used in rural highway design include a known and consistent subgrade, free draining granular layers, surface draining sideslopes away from the road structure, a known quality aggregate, and specified load limits. The highway structure design nomographs make use of equivalent single axle loadings (ESAL) to determine the design life of the roadway. Many of these design factors are not present in urban roadway systems and by using the design nomographs, urban roadway engineers are facilitating premature failure of the urban roadway network.

Within the last ten years, a significant increase in the amount of early structural failures of City of Saskatoon roadways has been observed (Berthelot et al. 2011). Recent research have shown that the new Local and Collector roadways are failing structurally at between 15 percent and 50 percent of their expected service life based on historic results (Prang, 2012). In order to determine the cause of the unexpected early failures, there is a need to undertake an evaluation of the design assumptions and structural design processes used by the City of Saskatoon.

In highway applications, drainage is accomplished by daylighting the granular layers into a drainage ditch beside the roadways (Saskatchewan Ministry of Highways and Infrastructure, 2009). Typical road construction in urban settings places the road structure in a “clay box,” leaving subsurface (bottom up or top down infiltration) moisture with no drainage path. In order for roadways to have a dry and therefore structural subgrade, construction has to be performed in high and dry areas, or there needs to be a free draining capillary break structure installed on top of the subgrade. As the City of Saskatoon is expanding, development is occurring increasingly in areas with wet subgrades (Prang 2012). Up until 1990 only 3.5 percent of the City was developed on marginal or wet subgrade soil. As the City expanded from 1990 to 2007, 37 percent of the new neighborhoods were in areas with low gradelines and wet subgrades. Projected development areas have approximately 70 percent of development in 2030 in wet subgrade areas. The future development map in appendix A indicates the future areas that the City of Saskatoon is planning on developing.

As roadways are reconstructed, City of Saskatoon inspection staff has noticed that excavated granular base aggregate has visibly more fines than expected (Ostrander, 2009). Visual inspection of other streets in the same areas have shown the presence of moisture containing fines being "pumped up" from the subgrade into the granular structure and then to the surface through cracks in the asphalt as shown in Figure 1.1.

Substructure drainage structures are the recognized method for mitigating substructure moisture infiltration in road structures (NCHRP, 1997). However, the cost of conventional drainage aggregates, primarily coarse crushed rock, has caused the City of Saskatoon to investigate alternative materials to use for the drainage structure (City of Saskatoon, 2009). Alternative aggregates used by the City of Saskatoon have included recycled asphalt as a base aggregate and recycled Portland cement concrete as a drainage aggregate (PSI Technologies Inc., 2010). These aggregates are not accounted for within City of Saskatoon specifications and design procedures and require a change to the design procedure and material specifications to be incorporated into normal use.

Freeze thaw cycles have been documented to result in a significant reduction in the strength of the roadway structure during critical periods in the spring and fall. Provincial Highway Departments have therefore typically implemented load restrictions in these periods to minimize damage. Weight restrictions are not feasible in an urban environment as enforcement is impractical given the typical grid and alternate route layouts common to urban centers. As well, commercial vehicles typically used in urban operations such as transit and waste removal operate year round on all road classes.

Figure 1.1 Moisture Pumping Out of the Roadway on Braeburn Crescent

The early structural failure of roadways combined with the diminishing quantities (Anthony, 2007) and quality of locally available quality virgin aggregates create the need to evaluate the design process and material types and specifications used in urban roadway construction. When determining the optimal life cycle user needs for urban roadway design, construction methods and end product road structure are key to future sustainability of road infrastructure assets in society.

The goal of this research is to investigate the use of recycled portland cement and asphaltic concrete rubble materials as a high quality aggregate system that improve roadway performance sustainability.

1.2 Research Hypothesis

It is hypothesized that asphalt and portland cement concrete rubble can be processed to a high quality aggregate road material that exceeds current conventional roadway aggregate material properties and requires less capital and life cycle cost.

1.3 Objectives

In light of the stated hypothesis of this research, the objectives of this thesis were as follows.

- Investigate the current City of Saskatoon road material specifications with respect to a mechanistic structural design perspective.
- Evaluate the technical feasibility of substituting recycled asphalt and portland cement concrete rubble aggregate materials in place of conventional City of Saskatoon pit run derived aggregate materials in an engineered structural base and drainage road structure application.
- Evaluate the technical effectiveness of using conventional and recycled rubble aggregates as an engineered drainage and strain dissipation substructure layer based on mechanistic road structural modeling and field structural validation measures.
- Evaluate the economic feasibility of using this recycled aggregate materials within the roadway structure.
- Provide mechanistic based road materials and structural design specifications that are applicable to all road building aggregate materials employed in the City of Saskatoon.

1.4 Scope

The scope of this research included the review of the existing City of Saskatoon roadway network and typical City of Saskatoon roadway design methods. Within the context of the City of Saskatoon design methods, the aggregate specifications and historic use of recycled aggregates was also reviewed. Conventional aggregates and recycled asphalt and portland cement concrete aggregates were characterized using conventional and mechanistic material tests.

Based on the characterizations performed, roadway designs were performed for local and arterial roadways using the conventional design method and a mechanistic design.

Test sections evaluated included sections of Marquis Drive, 115th Street, Kenderdine Road #1, Field House Road, 8th Street, Kenderdine Road #2, Wilkinson Crescent and Adolph Way. Structural evaluation of the test sections was done through the use of a falling weight deflectometer at primary highway weights.

Capital cost analysis was done using average costs incurred for aggregates through the 2011 construction season. An example of a possible life cycle cost was illustrated based on average capital treatment costs of 2011 for restoration treatments. As well, the timing for the treatments was based on expected life cycles of the various treatments as defined by the City of Saskatoon asset management system.

1.5 Methodology

The methodology of this research included the following elements and tasks.

Element 1. Literature Review and Background

- Task 1.1 Review of existing City of Saskatoon roadway network.
- Task 1.2 Review of typical City of Saskatoon design issues.
- Task 1.3 Evaluation of the City of Saskatoon aggregate supply and demand.
- Task 1.4 Review of the City of Saskatoon granular material specifications.
- Task 1.5 Literature review of traditional uses of recycled aggregate material in the City of Saskatoon.
- Task 1.6 Literature review of traditional uses of recycled aggregate materials in other similar agencies.

Element 2. Conventional Materials Characterization

Conventional material testing on identified graded materials was conducted. Each identified material considered in this research had samples taken for conventional testing and mechanistic – climatic durability characterization. Each sample was at least 22 kg of material to allow for repeat trials of the following tests:

Task 2.1 Sieve Analysis of Fine and Coarse Aggregates (ASTM C136).

Task 2.2 Test Method for Materials Finer than 75-m (No. 200) Sieve in Mineral Aggregates by Washing (ASTM C117).

Task 2.3 Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487).

Task 2.4 AASHTO Soil Classification System (ASTM D3282).

Task 2.5 Plasticity Index Test (ASTM D4318).

Task 2.6 Sand Equivalency Test (ASTM D2419).

Task 2.7 Coarse Aggregate Angularity Characterization (ASTM D5821).

Task 2.8 California Bearing Ratio Test (ASTM D1883).

Element 3. Mechanistic – Climatic Durability / Drainage Constitutive Material Characterization

Mechanistic-climatic material testing on identified aggregate materials was performed. Each identified material gradation will have samples taken for conventional testing and mechanistic – climatic durability characterization. Each sample was at least 22 kg of material to allow for repeat testing.

Task 3.1 Triaxial frequency sweep mechanistic characterization at room temperature. Samples were evaluated at four load frequencies and a stress state representative of typical City of Saskatoon field state conditions.

Task 3.2 Climatic durability conductivity and effect on mechanical behavior. This includes a moisture uptake test and a hydraulic conductivity test.

Element 4. Roadway Structural Design

Task 4.1 Conventional empirical City of Saskatoon roadway design on arterial, collector and local roadways using the City of Saskatoon Design Specifications was performed.

Task 4.2 Mechanistic roadway structural design of arterial, collector, and local roadways using material constitutive properties and theoretical numerical finite element structural analysis design.

Element 5. Field Structural Performance Validation

Task 5.1 Summary of test section construction including the design cross sections used and construction techniques utilized to complete the field test sections.

Task 5.2 Evaluation of test section performance through heavy weight deflectometer testing at primary weight loading level on all identified test sections. From the Heavy Weight Deflectometer (HWD) testing the peak surface deflections were determined for each test section.

Task 5.3 Validation of the test sections by comparing the non-destructive structural test results with the theoretical design.

Element 6. Evaluate Capital Cost and Long Term Sustainability Analysis

Task 6.1 Evaluation of Capital construction cost for the various structures identified in Task 4.1 and 4.2.

Task 6.2 Estimation of the structural design life for local and arterial roadways based on historic road structural performance asset management data in the City of Saskatoon.

Task 6.3 Evaluation of the drainage layer construction versus standard construction methods based on structural long term performance benefits.

Task 6.4 Evaluation of the design structures of the conventional and mechanistically modeled roadway designs in terms of energy used in the construction of the cross sections.

Element 7. Design and Specification Recommendations

Task 7.1 Outline the changes required in the City of Saskatoon specifications in order to incorporate recycled aggregates.

Task 7.2 Summarize the design elements required to incorporate a stress dissipating drainage layer in the roadway.

Task 7.3 Outline a design procedure based on mechanistic material properties for roadways in the City of Saskatoon.

Element 8. Research Summary, Conclusions and Future Recommendations

Task 8.1 Summarize the technical feasibility of substituting recycled material in the roadway.

Task 8.2 Summarize the net benefits of using a drainage layer within the roadway structure and the benefits of using recycled granular as this drainage structure.

Task 8.3 Summarize the requirements to integrate recycled materials into the design and construction of roadways within the City of Saskatoon.

Task 8.4 Summarize recommended future research topics.

1.6 Layout of Thesis

Chapter one provided the background information introducing the research topic of investigating the structural performance of alternate aggregate sources and its importance to the City of Saskatoon. This chapter presents the research goal, objectives, scope of the work, and the methodology performed to complete the research.

Chapter two summarizes the existing City of Saskatoon roadway network and design protocols. In evaluating the network and design protocols, issues relating to aggregate shortage and design shortfalls are identified. The current City of Saskatoon specifications are evaluated to show what qualities are required in aggregate materials. The literature review indicates how recycled rubble has been used in the past in Saskatoon as well as within other agencies around the world. Even though the general use of recycled aggregates is accepted, the recycled aggregates have not been incorporated in the roadway design as a valued component. This chapter outlines some of the accepted uses and detailed where the shortcomings in understanding are with recycled materials. This chapter also outlined the current City of Saskatoon roadway design procedure and the areas that have been shown to be deficient where a revised design methodology including recycled aggregates would better address the failure mechanisms.

Chapter three summarizes the material characterization values of the various aggregates, both recycled and virgin, that are used in building a roadway. The chapter focuses on the aggregate properties currently specified by the City of Saskatoon.

Chapter four summarizes the material characterization values of the various aggregates, both recycled and virgin, that are used in building a roadway from a mechanistic material characterization viewpoint. The mechanistic material characterization values used were designed specifically to simulate the current field state conditions experienced in the City of Saskatoon.

Chapter five compares two types of roadway designs. The first set of designs are the typical arterial, collector and local roadway designs currently specified by the City of Saskatoon assuming dry subgrade as well as what is currently required by the City of Saskatoon design manual. The design indicated the quantity, cost and type of aggregate for each road type. The second set of designs are theoretical designs based on the mechanistic characteristics of the aggregates and structural layer design. The theoretical designs also indicate the theoretical performance that should be expected of the roadway. Chapter five also evaluated the proposed changes to the City of Saskatoon specifications to determine if they will provide a roadway that meets the required performance. The evaluation was conducted by comparing the test data on the completed test sections against the performance requirements and the anticipated performance for each roadway design.

Chapter six summarizes the test sections that were constructed in 2009 and 2010 indicating their cross-sections and construction protocols. Each test section was then evaluated with non-destructive testing to determine if it met maximum deflection requirements set out by the City of Saskatoon asset management program.

Chapter seven summarizes the capital costs of the structures developed in chapter five. In addition to the capital costs, long term structural behaviour models developed in the City of Saskatoon have been used to evaluate the long term economic benefit of using drainage layers in locals and collectors versus not using any drainage. This chapter also outlines the energy requirements for the various cross sections using fuel usage as the measurement tool.

Chapter eight outlines the required changes for the City of Saskatoon specifications and design procedure in order to incorporate the recycled aggregate into the roadway design for the City of Saskatoon. As the recycled materials provide a better and more cost effective structure, the incorporation of the recycled aggregates into the design will help Saskatoon be more sustainable.

Chapter nine presents the summary and conclusions of the research. This summary includes recommendations for future research and suggestions for specification changes to implement the policy change.

CHAPTER 2 LITERATURE REVIEW

The City of Saskatoon has been experiencing roadways failing prematurely while constructing the roadways to standards that have been sufficient in the previous development construction. Examining the basis of the original construction design methods and specifications allows for an understanding of the existing conditions (City of Saskatoon 2011, City of Saskatoon 2008, Berthelot et al. 2011). When evaluating the design methods used for the poor subgrade areas that the City is currently expanding into, the usage of various pit run and recycled aggregates, and determining if structural failure is due to repetitive loading or a relatively small number of critical state loads in climatically sensitive periods is key in moving forward towards a more sustainable method to provide road infrastructure.

The City of Saskatoon spends over $30,000,000 annually on constructing and maintaining the roadway infrastructure. As this spending represents over 4.5 percent of the annual operating and capital budget for the City of Saskatoon, any optimizing or cost savings will have a significant impact on the citizens of Saskatoon.

2.1 Current Roadway Design Systems

There are a number of roadway design systems being used around the world. Roadway design systems are either empirical, mechanistic empirical or mechanistic based. Empirical design systems rely on either years of data or accelerated testing to determine a structure that will have a given performance over time. Mechanistic roadway design systems rely on knowing the mechanistic material properties of each of the roadway materials and using those material properties to predict the damage over time. Mechanistic-empirical roadway design use performance indicators and calibration factors to create a roadway structure that will meet performance guidelines

2.1.1 City of Saskatoon Roadway Design System

The City of Saskatoon roadway network was segmented into five classifications of roadways, depending on traffic loading, traffic volume and traffic type. The local classification was for residential streets and areas experiencing less than 1,500 ADT. Transit activity is prohibited on this road class for normal operations. The next larger designation is for Collector roadways where the roadway traffic volume is 1,000 to 12,000 ADT. The Industrial roadways designation is limited to industrial areas of the City and is specialized due to the higher volume of heavy truck traffic. Collector roadways feed into the City of Saskatoon Arterial roadway network. This network is designed for an ADT of between 5,000 and 30,000 ADT. The last road classification is for Freeway roadways designed for greater than 30,000 ADT.

Existing standards for roadway design in the City of Saskatoon are based on the Saskatchewan Highway Design Modified Shell Curve method (Saskatchewan Ministry of Highways and Infrastructure, 2009). The modified shell curves were developed by using a statistical regression analysis of the layered linear elastic primary responses of the roadway structures developed and constructed in the AASHTO road test. In using this model, Saskatchewan was one of the first adopters of a mechanistic – empirical design in roadways. The design uses resilient modulus values of the materials and then correlates the resilient modulus to CBR values for the roadway aggregates and subgrades. This system then correlates a soaked CBR value of subgrades and granular types with a required thickness of asphalt in order to keep the vertical compressive strains occurring at the top of the subgrade below a specified maximum to ensure long term performance as well are ensuring the tensile strain at the bottom of the asphalt layer is below a specified maximum. Shear strains are not identified or considered in this design method. The modified shell curve also relies on a fully compacted and prepared subgrade which is not practical in urban rehabilitation due to user demands and time requirements.

In order to design with this method various parameters need to be known. The number of equivalent single axle loads (ESAL), soaked CBR of the subgrade soil as well as the granular unsoaked CBR of the materials available are used to determine the thickness of the various granular layers and asphalt surface. An example of a typical shell curve to design structure thicknesses is shown in Figure 2.1. The typical structure for a City of Saskatoon local roadway is identified in the figure.

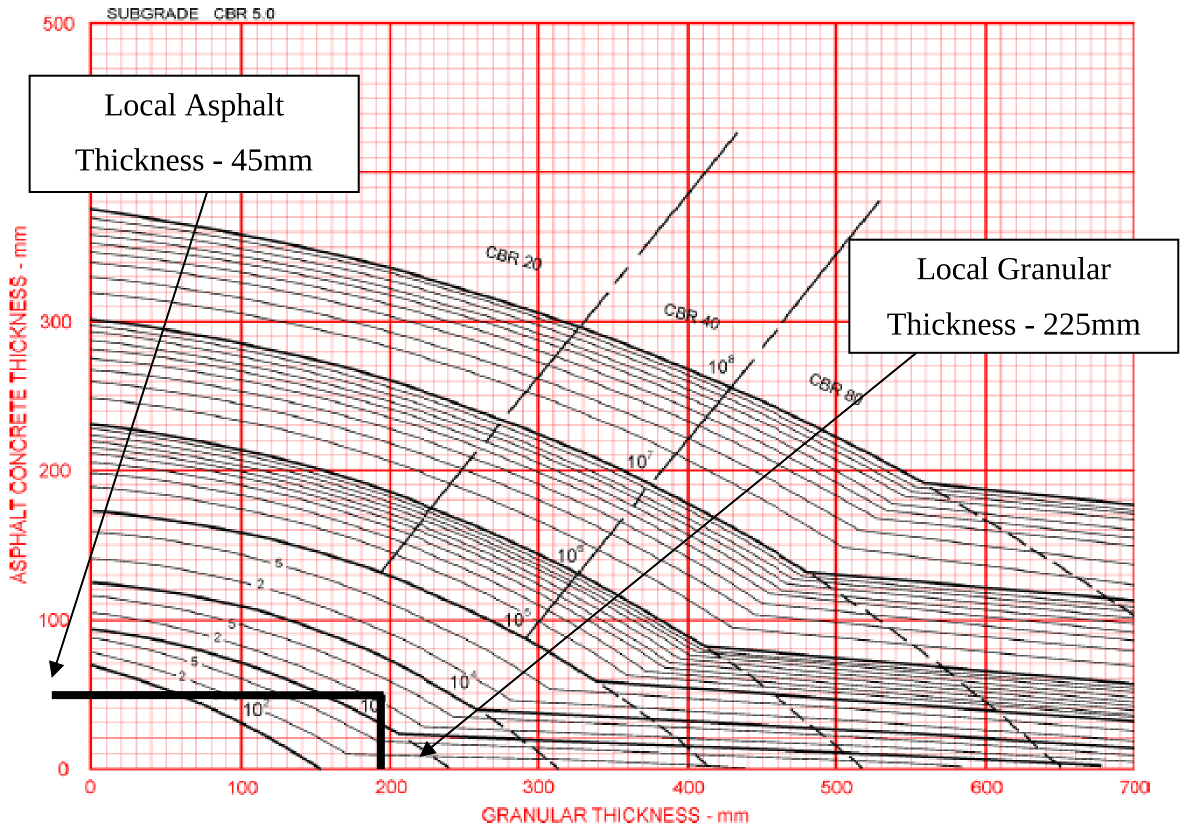


Figure 2.1 Shell Curve Example for CBR of 5.0

Given the above shell curve with a CBR subgrade of 5, using the design thickness for a local roadway at 45mm of asphalt and 225mm of granular, the design ESALs would be approximately 5x10^3 for a 20 year design life. While many local roadways do not reach even this low number of ESALs, the structure is more prone to failure under a critical load state and critical climatic conditions experienced in the City of Saskatoon. The standard method of adding an extra 150 mm of base for subgrades with a CBR of <5.0 matches up with the shell curves as shown by Figure 2.2.

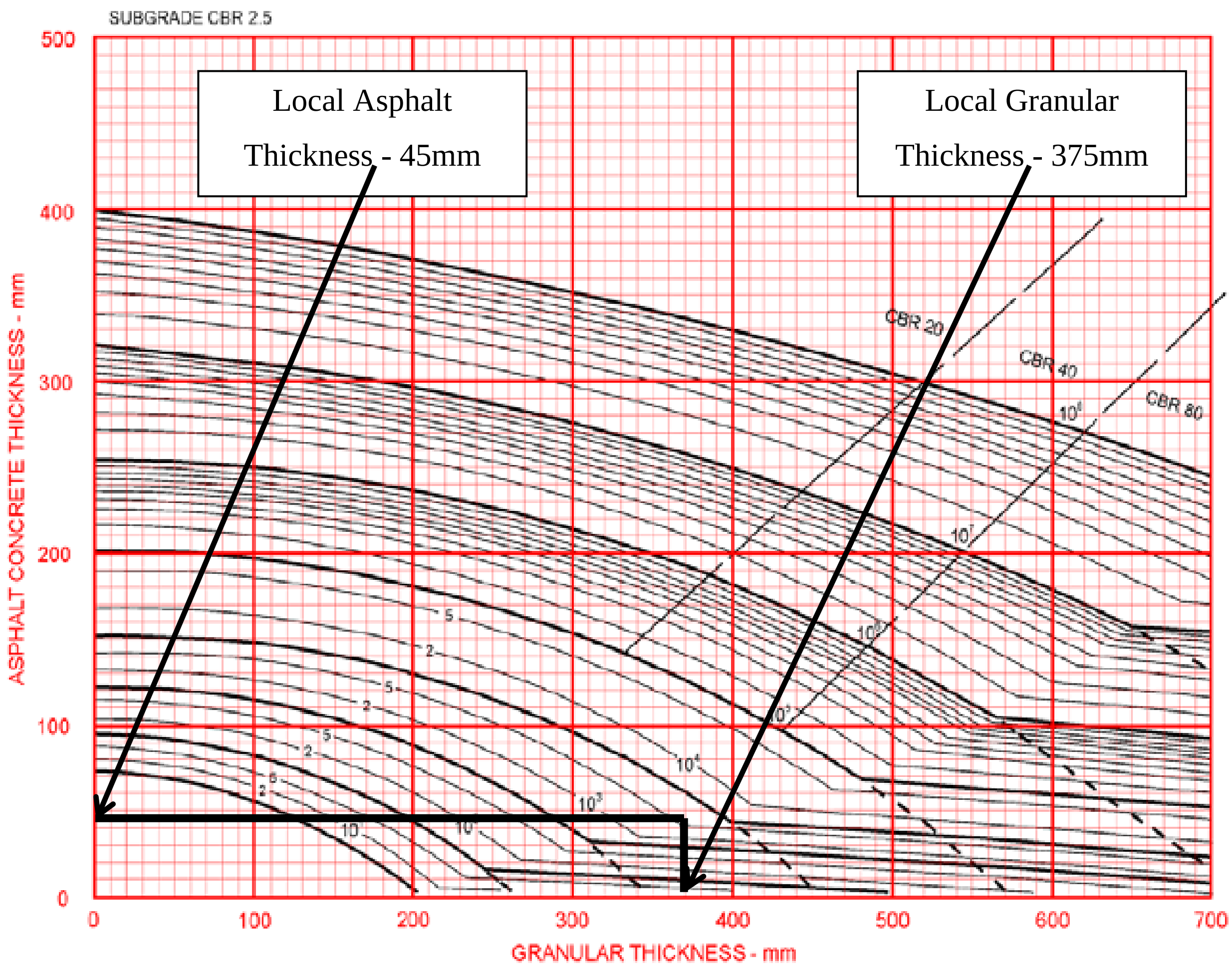


Figure 2.2 Shell Curve Example for CBR of 2.5

In the design of the shell curve, the granular materials are identified as having an unsoaked CBR of 20, 40 and 80 while the COS specifies granular material of having a minimum unsoaked CBR of 25 for sub-base and 65 for granular base. The COS does not correct for actual CBR of the granular but rather assumes the granular to follow the curves identified in the SMHI shell curves.

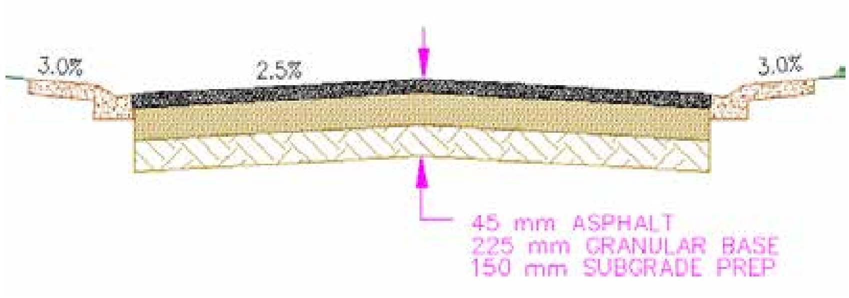
Due to these shell curves a standardized road structure thickness has been determined for the roadway network for all areas based on whether their subgrade CBR is greater than five as shown in Table 2.1.

Table 2.1 Roadway Thickness Structures for City of Saskatoon

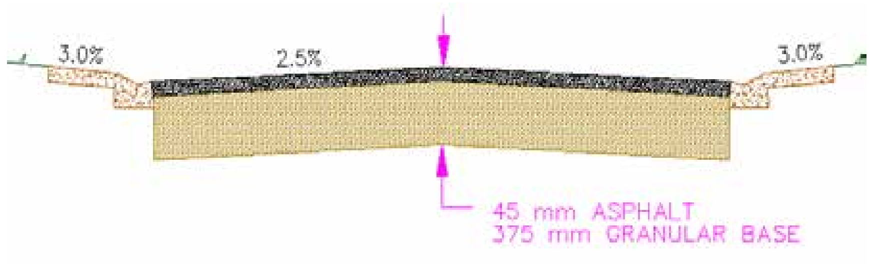
	Layer Thickness (mm)		
	Local	**Collector**	**Arterial**
Subgrade CBR >5			
HMA	45	80	100
Base	225	150	150
Sub-base	0	225	300
Subgrade Prep	150	300	300
Subgrade CBR<5			
HMA	45	80	100
Base	375	150	300
Sub-base	0	375	300
Subgrade Prep	0	0	0
Geotextile	no	no	yes

As outlined in the neighbourhood planning guide (City of Saskatoon, 2008), the City of Saskatoon will increase the thickness of the granular structure by 150mm in place of 150mm of subgrade preparation if the subgrade has a CBR of less than 5.0. This procedure is done across all road classes except expressways. These granular structures will vary from as low as 225mm for local roadways up to 450mm for the standard arterial structure. Each structure in a low subgrade CBR situation would have an additional 150mm of granular structure. The cross sections of the design local and arterial structures as outlined in Table 2.1 are illustrated in Figure 2.3.

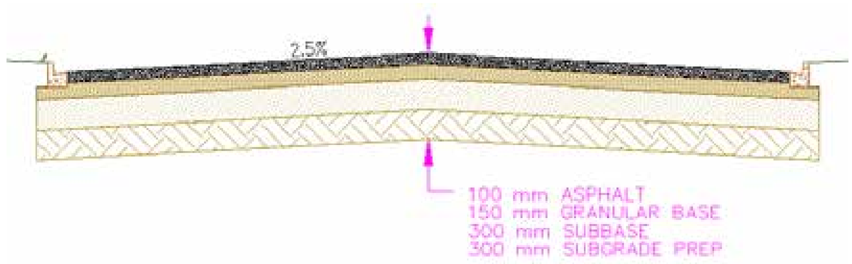
At traffic levels less than 10^5 ESALs, the layer thicknesses identified for the different CBR granular materials becomes less than 100mm. As 100mm is the minimum thickness for a granular layer in the roadway, all roadways constructed for traffic levels less than 10^5 are assumed to be constructed with one granular layer only. The SMHI designates the various CBR granular materials at 20 (Type 6 sub-base), 40 (Type 35) and 80 (Type 33) while the COS only has the two granular designations in roadway construction of CBR 25 (sub-base) and 65 (base). However, as this was the most common local design system when the City of Saskatoon Specifications was developed, City road structures were designed using the asphalt and aggregate thicknesses identified in these nomographs.



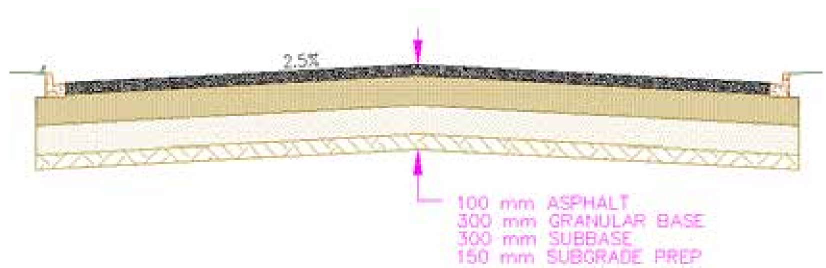
a) Typical COS Local Structure with Dry Subgrade



b) Typical COS Local Structure in Wet Subgrade



c) Typical COS Arterial Structure with Dry Subgrade



d) Typical COS Arterial Structure with Wet Subgrade

Figure 2.3 Typical COS Structures

2.1.2 AASHTO Mechanistic - Empirical Roadway Design System

One of the most common empirical design methods is the 1993 Association of the American State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures. This design method uses the observations of the Association of American State Highway Officials (AASHO) Road Test that was completed in Ottawa, Illinois from 1956 to 1962. A test track was constructed with test sections of varied thicknesses of roadway aggregates and surfacing materials. Both rigid and flexible roadway systems were constructed and evaluated using accelerated traffic loading on the road test track. Physical measurements of the roadway sections such as rut depth, cracking were completed periodically. These measurements allowed for a design to be quantified by a structural number (SN) for the amount of load applications required to produce a specified serviceability loss.

While the AASHO test produced a design system that correlated well with the performance observations, there were numerous limitations to the scope of the AASHO road test (Transportation Research Board Pavement Management Section, 2007) (Pavement Interactive, 2007). The limitations included only testing one subgrade type, one set of environmental conditions, one set of roadway material types and one tire pressure. As most of these conditions vary by location and have changed over time, this design methodology required changes.

The Mechanistic Empirical Performance Design Guide (MEPDG) was created to update the AASHTO Guide to address the varied conditions with newer and more complete roadway performance data that had been collected with long term testing in different regions of North America (Li, Xiao, Wang, Hall, & Qiu, 2011). This design system has adopted the use of resilient modulus testing of subgrades and roadway aggregates instead of CBR.

Many provinces have adopted the MEPDG or are using a variation of the AASHTO design (Transportation Association of Canada, 1997) (Canadian Strategic Highway Research Program, 2002) that has been empirically calibrated to their regional materials and climatic effects. It is common practice for the cities to adopt the same design practices of their respective province due to a limited amount of resources available to do the design.

Currently the mechanistic test parameter most prevalent in roadway structural design is the resilient modulus. The resilient modulus is determined by applying a haversine load to the material finding the stiffness of the material under a simulated traffic loading. The resilient modulus also provides the amount of permanent and residual strain in the aggregate structure from repeated loading (Kim & Labuz, 2007). Typical resilient modulus values of aggregates range from 200 MPa to 400 MPa depending on the confining and deviatoric stress. When the same range of confining and deviatoric stresses are placed on a 25 % conventional aggregate – 75% RAP blend the resilient modulus ranged from 250 MPa to 500 MPa, approximately a 25% increase in stiffness.

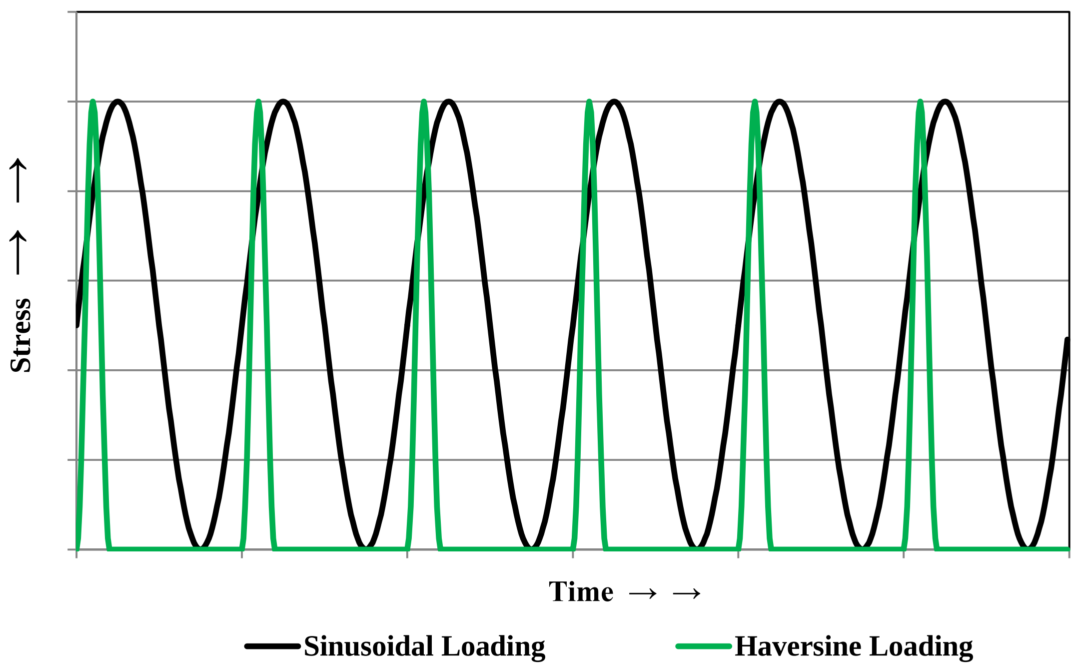


Figure 2.4 Comparison Between Sinusoidal Loading and Haversine Loading

This thesis investigated the use of dynamic modulus instead of resilient modulus due to the information that is provided in dynamic modulus testing. Dynamic modulus testing evaluates the aggregates using different frequencies to determine the varying effect of traffic speed on the strains and therefore determining the viscoelastic properties. Dynamic modulus testing also uses a sinusoidal loading instead of the haversine loading used in resilient modulus. Figure 2.4 illustrates the difference in load form between the haversine and sinusoidal loading. The haversine loading of a 0.1 second load followed by a 0.9 second rest period is used to simulate loads as would be experienced with axle groups on vehicles. The rest period provides information on the recovered strain between loading. The advantage of the sinusoidal loading is the ability to determine the lag between the peak stress and peak strain which enables the determination of the materials linear viscoelastic properties.

2.2 Mechanistic Design of Roadways

Road granular materials are used in high deviatoric and highly dynamic loading field state conditions. It is therefore imperative that road materials be evaluated in a similar framework in order to determine the roadway structural capacity. One of the tools used to evaluate the materials in this manner is the Rapid Triaxial Testing (RaTT) Apparatus shown in Figure 4.1. This testing apparatus has the ability to cyclically apply variable loads in all three axes.

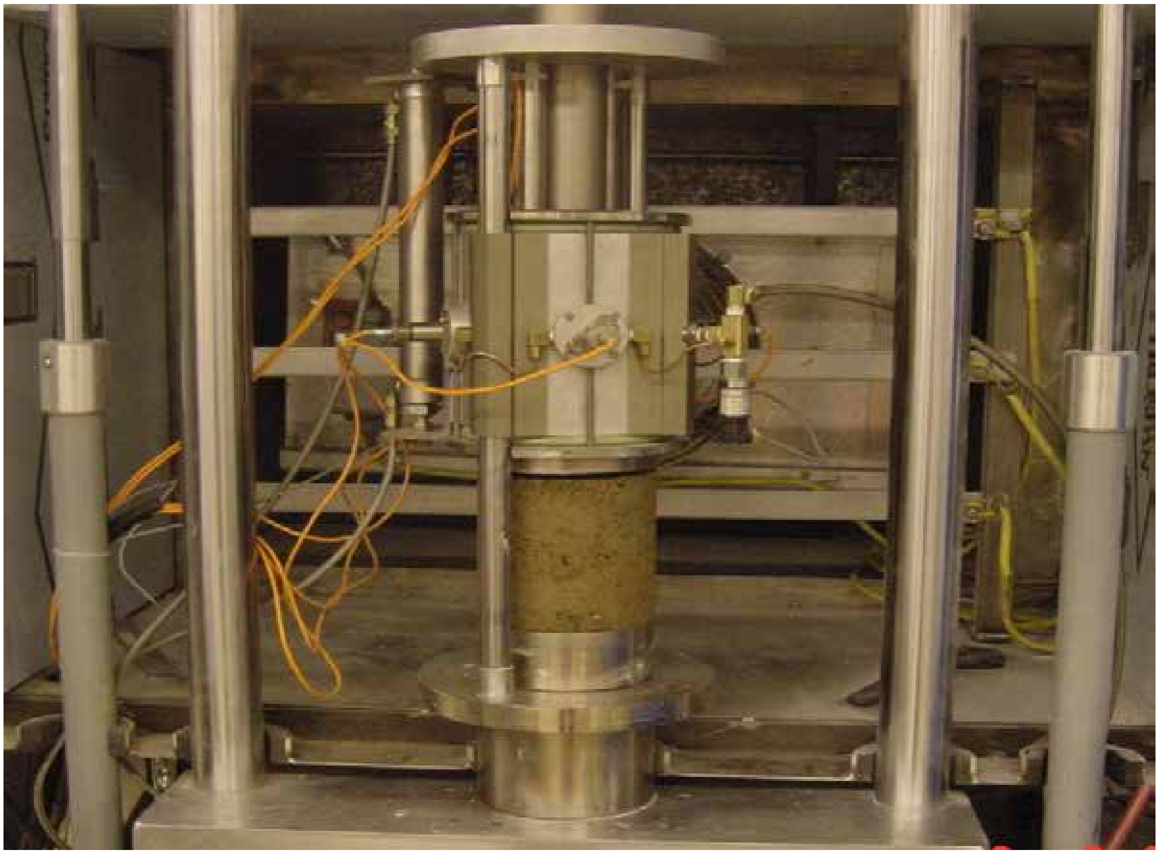


Figure 2.5 Rapid Triaxial Testing Apparatus

2.2.1 Mohr – Coulomb Failure Envelope

The two main components that are commonly used to evaluate the shear characteristics of soils are cohesion and internal friction (Holtz & Kovacs, 1981). As indicated in Figure 2.6, the Mohr - Coulomb failure envelope theory determined that the cohesion will determine the strength of the soil when there is no external stress while the internal friction will be enacted as stress is applied. Materials such as clay tills will have a high cohesion but a low slope due to little to no internal friction while crushed rock will have a lower intercept (cohesion) and a steep slope due to high internal friction values. Cohesion is driven by the amount of fine materials that "stick" to each other. Fracture content and toughness of the aggregate will determine the friction angle. A highly fractured aggregate will have high internal friction as long as the aggregate does not easily fracture under stress.

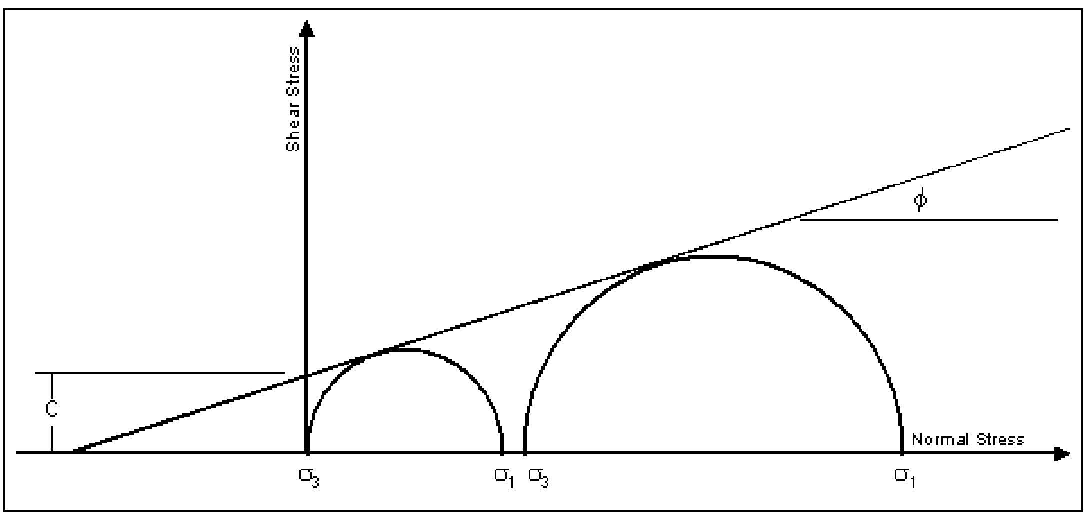


Figure 2.6 Mohr-Coulomb Failure Envelope

(http://www.fhwa.dot.gov/publications/research/safety/04094/04.cfm)

The formula for the Mohr-Coulomb failure criteria is as follows:

- $\tau = \sigma \tan \emptyset + c$ [2.1]

where:

τ = shear strength

σ = normal stress

φ = angle of internal friction

c = cohesion

Typical roadway aggregates will undergo significant normal stresses. In order to prevent shear failure, the aggregate must remain under the failure envelope as determined through the Mohr-Coulomb theory. Standard base aggregates will contain a mix of some clay fines to introduce cohesion as a binder as well as a significant fracture content to raise the friction angle. However, when the aggregate mix includes too many clay fines, the cohesion value may be significantly reduced, even dropping below zero in the presence of high moisture contents. Therefore, when constructing roadways with granular base that includes clay fines, it is critical to either remove the presence of water or decrease the presence of the clay fines. As the recycled asphalt and portland cement concrete aggregates had low content of plastic fines and high fracture counts the risk of shear failure in the roadway is significantly reduced when constructing with these recycled aggregates.

2.2.2 Dynamic Modulus

The dynamic modulus is defined as the absolute value of the complex modulus. As shown in the expression below, the complex modulus, (E*) is the ratio of the amplitude of the sinusoidal stress applied to the amplitude of the resultant sinusoidal strain (Berthelot 1999).

$$E^* = \frac{\sigma}{\varepsilon} = \frac{\sigma_{11p} e^{i\omega t}}{e_{11p} e^{i(\omega t - \delta)}} \qquad [2.2]$$

where:

E* = Complex Modulus (Pa)

σ = Applied stress (Pa)

ε = Strain response to applied stress (µm/µm)

σ_{11p} = Peak stress applied in the X_1 coordinate direction (Pa)

e = Exponent e

i = Imaginary component)

ω = Angular load frequency (radians per second)

t = Load duration

ε_{11p} = Peak strain response in X_1 coordinate direction (µm/µm)

δ = Phase angle (radians)

Therefore the formula for the dynamic modulus can be shown as follows:

$$E_D = |E^*| = \frac{\sigma_{11p}}{\varepsilon_{11p}} \quad [2.3]$$

where:

E_D = dynamic modulus

E* = Complex Modulus (Pa)

σ_{11p} = Peak stress applied in the X_1 coordinate direction (Pa)

ε_{11p} = Peak strain response in X_1 coordinate direction (µm/µm)

Aggregate materials each respond uniquely to stresses at different frequencies and moisture contents. This information can be gathered to determine how the material will behave in specific climatic conditions under critical load states. The design of the roadway structure needs to take into account the variability of the traffic speeds and loading throughout the roadways life.

2.2.3 Poisson's Ratio

Poisson's Ratio is defined as the relationship between the lateral strain and the axial strain. This relationship can be measured and evaluated throughout the various stresses that the rapid triaxial testing apparatus uses. The ratio can be expressed as (Berthelot 1999).

$$v(t) = \frac{\varepsilon_{22}(t)}{\varepsilon_{11}(t)} = \frac{\varepsilon_{33}(t)}{\varepsilon_{11}(t)} \quad [2.4]$$

where:

υ (t) = Poisson's Ratio

t = Load duration (seconds)

$\varepsilon_{22}(t)$ = Lateral Strain in X2 coordinate direction (µm/µm)

$\varepsilon_{33}(t)$ = Lateral Strain in X3 coordinate direction (μm/μm)

$\varepsilon_{11}(t)$ = Axial Strain in X1 coordinate direction (μm/μm)

2.2.4 Complex Shear Modulus

The complex shear modulus is defined as the ratio of the shear stress to the shear strain and is shown by the following formula.

$$G^* = \frac{E^*}{2*(1+\upsilon)} \qquad [2.5]$$

where:

G* = Complex Shear Modulus (Pa)

E* = Complex Modulus (Pa)

υ = Poisson's Ratio

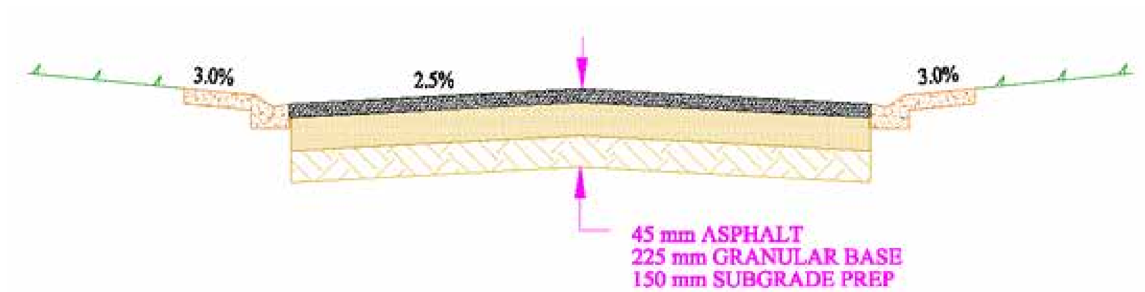
2.3 Existing City of Saskatoon Roadway Network

The City of Saskatoon roadway network, as of 2010, consists of nearly 12,300,000 square metres of roadways split up into four distinct road classes as shown in Table 2.2. Each of these road classes is designed with a different structure based on the amount and type of traffic loading expected over its service life.

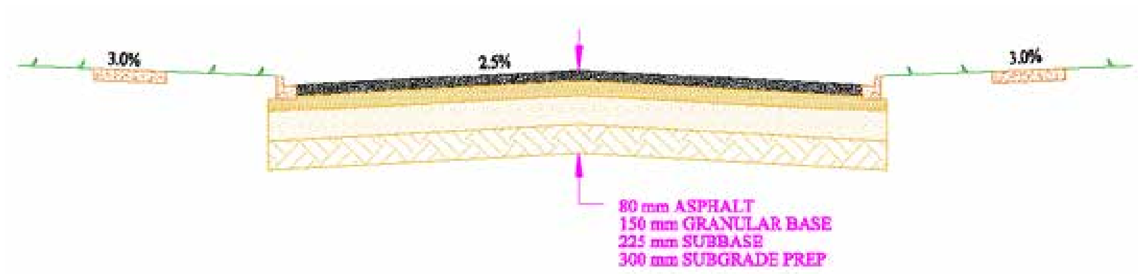
Table 2.2 Existing City of Saskatoon Roadway Areas by Classification

Road Classification	Area (sq.m.)	% of total	ADT
Locals	7,059,031	57%	<1,500
Collectors	1,990,116	16%	1,000-12,000
Arterials	2,408,263	20%	5,000-30,000
Expressways	842,351	7%	>30,000
Total	12,299,761	100%	-

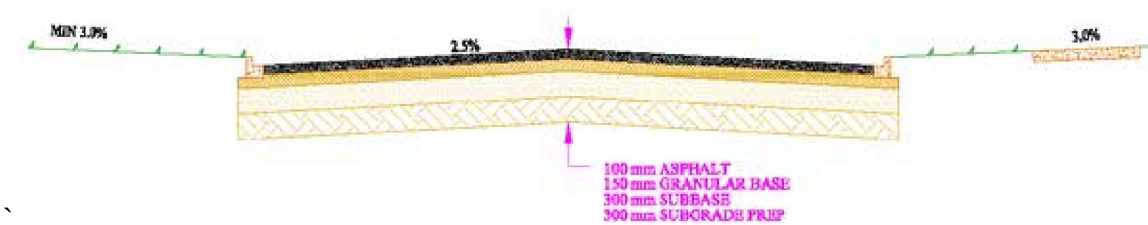
Construction of new roadways, at the rate of 2.3 percent growth per year, is still performed using these same design protocols. Over 300,000 square meters of roadway is constructed every year using structural designs not intended for urban use. Each of the City of Saskatoon roadway structures, as typically constructed in the City of Saskatoon, are shown below in Figures 2.2 with the exception of the expressways (City of Saskatoon, 2008). As the expressways are designed based on expected life cycle ADT, each design is unique to the section of expressway.



a) Typical Local Roadway Cross Section



b) Typical Collector Roadway Cross Section



c) Typical Arterial Roadway Cross Section

Figure 2.7 Typical City of Saskatoon Roadway Cross Section

As shown in Figure 2.7, aggregates make up approximately 83 percent by volume of the roadway structure. Within the roadway budget, the granular layers account for 34 percent to 45 percent of the cost. Therefore, in order to have adequate structural roadway, the City of Saskatoon is relying significantly on the aggregate portion of the structure. However, as the sources and quality of aggregate diminish, the structural capacity of the roadways will also diminish if steps are not taken to implement design changes.

2.4 Review of Typical City of Saskatoon Design Issues

Over the last 30 years there have been a number of road failures as early as five years into the roadway performance life. Invariably, failure is directly related to an insufficient design for the poor subgrade as well as a requirement for subsurface drainage. Figure 2.8 illustrates a local road in City of Saskatoon experiencing moisture issues. While the moisture issues do not seem critical to the residents, the roadway will soon develop severe cracking and fail as shown in Figure 2.9.

Figure 2.8 Moisture Issues on Braemar Court

The street shown in Figure 2.8 was constructed in 1991 and is showing signs of structural failure with the water pumping up as well as the beginnings of structural failures propagating to the surface. While the COS design protocols call for using the standard design only when the subgrade CBR is greater than 5.0, many roadways were constructed in with the standard structure on subgrades less than 5.0 due to engineer inexperience and cost saving measures. The water pumping is a symptom of many roadways in wet subgrade areas within the City of Saskatoon. Upon reconstructing many failed roadways, the granular structure was found to be wet and is contaminated with subgrade fines (Ostrander, 2009). Inspection of other roadways within these neighbourhoods has shown similar signs of moisture pumping in the granular structure. Figure 2.10 shows the same segment with non-destructive testing. Deflection points indicate that the roadway is flexing well into the failure envelope of roadways. Increased fines within the base aggregate result in lower structural capacity of the base aggregate (Guenther, Haichert, Foth, & Berthelot, 2011).

Figure 2.9 Failed City of Saskatoon Roadway Due to Substructural Moisture

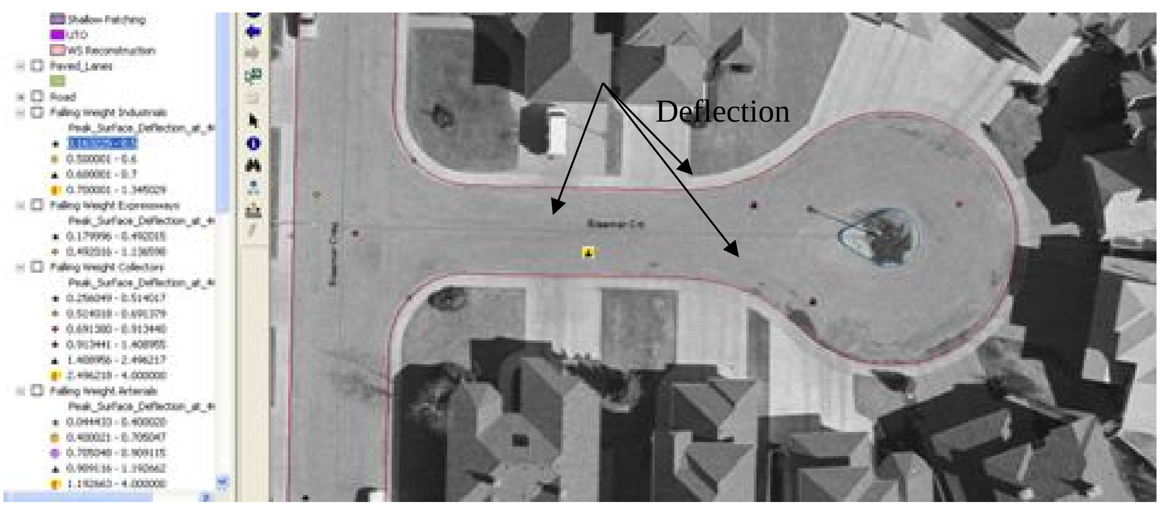


Figure 2.10 Non-Destructive Data as Shown by COS Mapguide (Braemar Ct.)

The City of Saskatoon has been changing their asset management system from simply identifying surface defects to a more holistic structural asset management system (Prang 2012). In doing so, non destructive testing has been used to evaluate the entire granular structure as an operating system. Non destructive testing can identify the failed portion prior to the visual signs than typical roadway asset management tools employ. By identifying problem areas early, steps can be made to minimize the costs required to maintain, repair and replace roadway sections affected by subsurface water.

A portion of the City of Saskatoon local and collector roadway network have been HWD tested at primary weights. When cross comparing the deflections with the age of the roadway, there is a clear correlation between subgrade type and field performance of the roadway over time as shown by Figure 2.11 and Figure 2.12. Roadways constructed in poor subgrades had higher deflections than good subgrade roadways. As well, high deflections occurred earlier in the roadways service life.

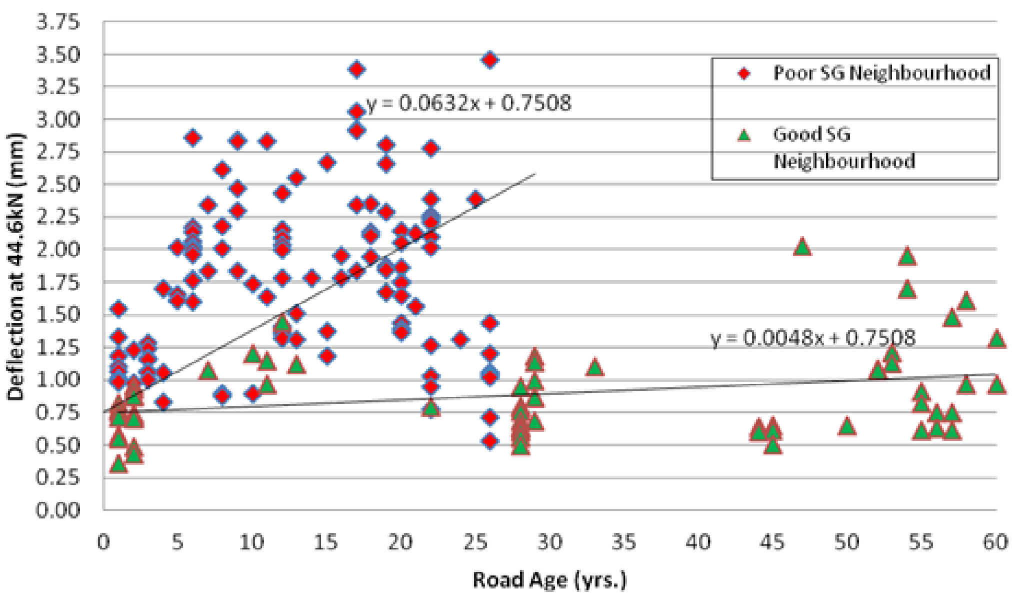


Figure 2.11 Deflection Versus Age for Local Roadways in the City of Saskatoon (Prang 2012)

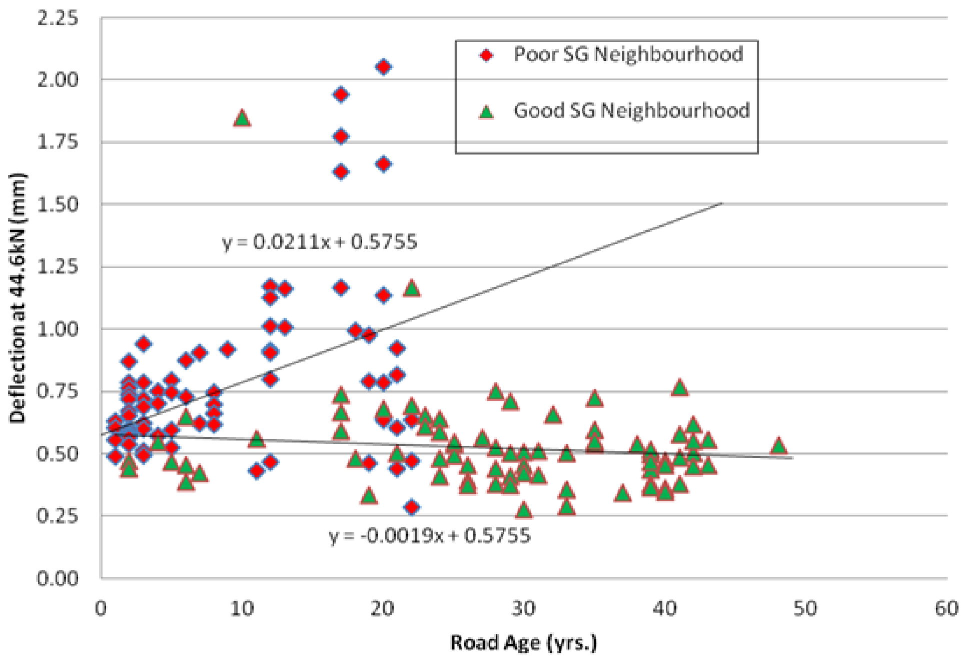


Figure 2.12 Deflection Versus Age for Collector Roadways in the City of Saskatoon (Prang 2012)

As illustrated in Figure 2.11 and Figure 2.12, there is correlation between the subgrade in the neighborhood and the amount of deflection experienced through the field structural tests. By quantifying in explicit field structural measures, preferably identifying problem areas before they have structurally failed, funding can be put in place *a priori* to replace the structures with self-draining road structures before catastrophic failure. Once structural failures have been identified, finding the best and most economical means of dealing with the structural failure is the next step.

As the City of Saskatoon continues to grow in population and infrastructure, some of the new neighbourhoods are situated in areas that previously were low lying areas with a high water table. The typical subgrade soils in these areas are silts and silty clays (Prang 2012). Constructing roadways in these areas requires an understanding of subsurface drainage as well as available materials to provide the drainage and determining the characteristics of the subgrade. The design manual currently used by the City of Saskatoon (City of Saskatoon, 2008) does not have any provisions for the design of drainage layers nor any post construction method for evaluating the structural primary responses. Therefore constructing roads in these areas is typically done without subsurface drainage assuming that the road cross section is sufficient to carry the loads.

Since the City of Saskatoon has based its roadway structures on a highway design with a high self-draining rural cross section and constructed it in an urban environment without sideslopes, substructure drainage becomes very important. However, in an urban environment, there is typically no exit for water trapped in the granular structure unless a drainage structure is constructed in the roadway or the subgrade is free draining. Without a means for an outlet for the water a clay box is created around the granular structure of the roadway. The excess water then is trapped in the subsurface granular structure significantly reducing its serviceable life and structural capacity. Research shows that over time, especially without the free draining granular layers, the base, sub-base and subgrade will wet up and lose structural capacity (Crystal Lacher, 2006).

There are many alternatives for subsurface drainage that have been attempted in the City of Saskatoon (Berthelot et al. 2011). The most common is some form of drainage aggregate layer underneath the road base and/or sub-base with edge drains bringing the subsurface water to the storm sewer system through the catch basins. Removing the water from the subgrade through the catch basins as shown in Figure 2.13 is key in retaining the structural capacity of the granular and subgrade materials.

The drainage aggregate used by the City in the past has been crushed portland cement concrete, sand and crushed rock. In most cases, the thickness of structure on top of the drainage layer was equivalent to the normal structure of that road class because the strain dissipation capabilities of the drainage layer were not considered in the structural design.

The use of different materials needs to be evaluated from a holistic view to determine their material characteristics, costs and availability for construction. As well, a means to incorporate these innovative road materials into the current City of Saskatoon specifications and general use within the City of Saskatoon needs to be developed.

Figure 2.13 Typical Connection of Drainage Layer to Storm Sewer System

2.5 Evaluation of the City of Saskatoon Roadway Aggregate Needs

There are a number of identified roadway needs in the City of Saskatoon. There is a growing shortage of high quality aggregates within the immediate area of the City of Saskatoon. As aggregate sources get depleted contractors are required to go further away from the City to mobilize in the required aggregates. This increased haul distance translates into increased energy and costs for the delivery of the aggregates as well as increased wear and tear on the roadway network to transport the material over the larger distances. To illustrate, Figure 2.14 indicates the current locations from which aggregates are being hauled to construct roads in Saskatoon.

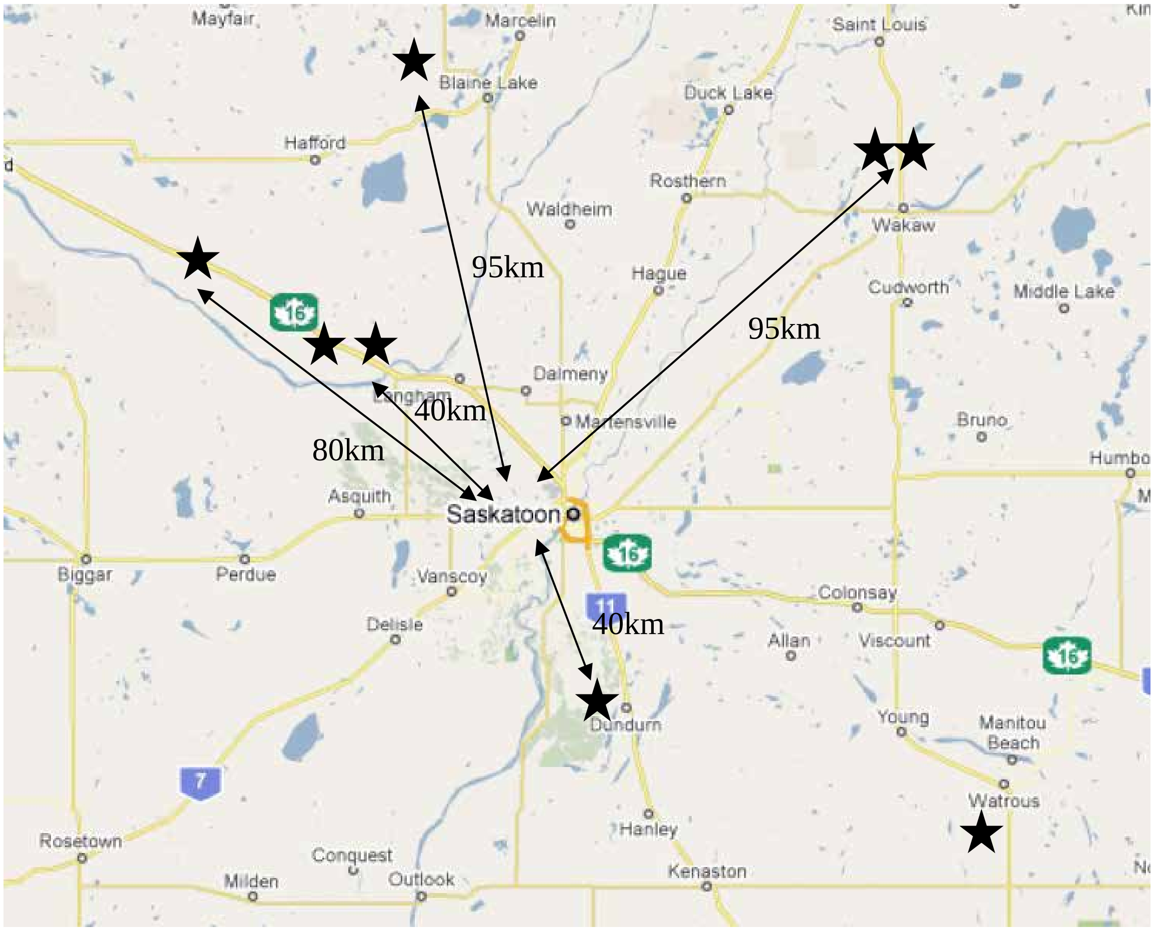


Figure 2.14 Pit Locations Around the City of Saskatoon

With the closest pits from which the City of Saskatoon receives aggregates being at least 40km away from the City and upwards of 100 km of haul distance for various aggregates the City of Saskatoon is experiencing an increasing cost to use glacial aggregates. The typical haul cost for aggregates in Saskatchewan is \$0.25/tonne-km.

2.6 City of Saskatoon Roadway Material Specification Review

Current City of Saskatoon granular specifications are based upon evaluating the physical properties of the granular material and subgrade properties as well as various empirical mechanical behavior values that can be related to historical results of comparable aggregates. The values required by the city are outlined in division three of the specifications and are used for all aggregates within the road structure (City of Saskatoon, 2011).

2.6.1 Soil Classification

There are two standard methods to classify road aggregates and soils. The first system is the American Association of State Highway and Transport Officials (AASHTO) soil classification system as shown in Table 2.3

Table 2.3 AASHTO Soil Classification System

General Classification	**Granular Materials (35% or less passing the 0.075 mm sieve)**						
	A-1		**A-3**	**A-2**			
	A-1-a	**A-1-b**		**A-2-4**	**A-2-5**	**A-2-6**	**A-2-7**
Sieve Analysis, % Passing							
2.00 mm (No.10)	50 max	…	…	…	…	…	…
0.425 (No.40)	30 max	50 max	51 min	…	…	…	…
0.075 (No.200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max
Characteristics of Fraction Passing 0.425 mm (No. 40)							
Liquid Limit	…		…	40 max	41 min	40 max	41 min
Plasticity Index	6 max		N.P.	10 max	10 max	11 min	11 min
Usual types of significant constituent materials	stone fragments, gravel and sand		fine sand	silty or clayey gravel and sand			

This system was first developed by Hogentogler and Terzaghi in 1929 but has been revised since then by AASHTO. The goal of this system is to provide a standardized system of evaluating soils for the purpose of constructing roadways across jurisdictions. As the name implies, this system is primarily used in the United States but is also used by some agencies in Canada.

The second method of evaluating the soils is the Unified Soil Classification System (USCS). The means of evaluating the soils is shown in Table 2.4. This system is used to describe the texture and grain size of the various soils and aggregates.

Table 2.4 Unified Soil Classification System

Major Divisions			Group Symbol	Group Name
Coarse grained soils more than 50% retained on No. 200 (0.075 mm) sieve	gravel > 50% of coarse fraction retained on No. 4 (4.75 mm) sieve	clean gravel <5% smaller than #200 Sieve	GW	well graded gravel, fine to coarse gravel
			GP	poorly graded gravel
		gravel with >12% fines	GM	silty gravel
			GC	clayey gravel
	sand $\geq$ 50% of coarse fraction passes No.4 sieve	clean sand	SW	well graded sand, fine to coarse sand
			SP	poorly-graded sand
		sand with >12% fines	SM	silty sand
			SC	clayey sand

2.6.2 Roadway Material Physical Qualities

When characterizing roadway materials the materials are classified in the City specifications as sub-base, base, crushed rock, and pit run (City of Saskatoon, 2011) and are shown in Appendix B. The characteristics of the various aggregate materials are usually predominated through gradation, fractured face count and unsoaked CBR. Gradations for various aggregates are identified by Figure 2.15 and Table 2.5 below.

Table 2.5 City of Saskatoon Aggregate Gradation Specifications

Sieve Size (mm)	Base		Sub-base		Crushed Rock	
	Min	Max	Min	Max	Min	Max
50	100	100		100		100
25	100	100	75	100	0	80
18	87	100				
12.5	72	93	52	100	0	18
5	45	77	30	75	0	12
2	29	56	20	55		
0.9	18	39				
0.4	13	26	8	30		
0.16	7	16				
0.071	6	11	3	15	0	5

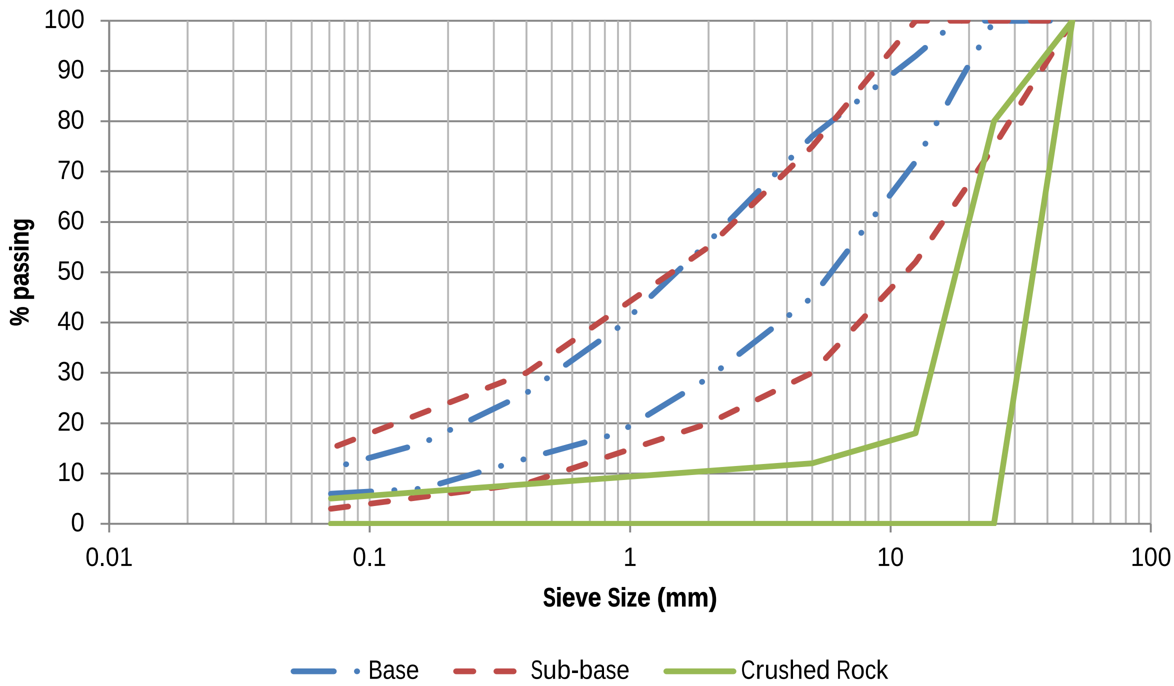


Figure 2.15 City of Saskatoon Aggregate Gradation Specifications

2.6.2.1 Sub-base Specifications

The City of Saskatoon typically has two granular materials specified for use within the roadway structure (City of Saskatoon, 2011). The granular material most typically designed to be on top of the subgrade is what is termed the sub-base. Sub-base is typically considered to be a poorer quality granular material which supports the granular base structure. Sub-base aggregate is further specified as having:

- Plasticity Index of material passing the 400 µm sieve shall not exceed 6.
- Specific Gradation as shown in Table 2.4.
- Organic content of material passing the 5mm sieve shall not exceed 3 percent by weight.
- Unsoaked CBR of 25 and greater.

Recycled aggregate is currently neither specified nor excepted in the specification for sub-base aggregate. Its use as a sub-base has been accepted on projects by the City of Saskatoon provided it meets the above characteristics of sub-base.

2.6.2.2 Granular Base Specifications

The second granular layer typically used in roadway structural design according to the City of Saskatoon specifications is termed as base aggregate (City of Saskatoon, 2011). The base aggregate is typically the higher quality granular material that forms the support for the asphalt surface. Base aggregate is specified as crushed gravel, sand filler and clay binder with:

- Specific gradation as shown in Table 2.4.
- No greater than 1 percent by weight organic content.
- At least one fractured face on at least 50 percent of material larger than 5mm.
- Unsoaked CBR of 65 or greater.

2.6.2.3 Crushed Rock Specifications

Crushed rock, while shown in the City of Saskatoon specifications (City of Saskatoon, 2011) is not typically used within current road construction design (City of Saskatoon, 2008). The specifications given are for material used around the pipe in typical water and sewer construction activities. However, crushed rock can also provide for a quality material to be used as a structural drainage layer within a roadway. Crushed rock is specified as fragments of durable rock free from soft or flaky particles, shale, loam or other deleterious material with:

- Specific gradation as shown in Table 2.4, and
- At least one fractured face on at least 50 percent of material larger than 5mm.

2.6.2.4 Pit Run Specifications

Pit run, while specified within the City of Saskatoon specifications, is not typically used within current road construction design (City of Saskatoon, 2008). However, the granular material is still available and has been used, in a limited quantity, as filler in over excavated areas during roadway construction. Pit run is specified as durable aggregate free from topsoil with:

- Top size of 150mm.
- At least 35 percent material by weight larger than 5mm.

2.6.2.5 Recycled Aggregate Material

Recycled aggregates have been used in varying degrees for some time as a substitute aggregate in roadways in place of virgin aggregate (Foth et al., 2011) (PSI Technologies Inc., 2010). In doing so, the vast majority of agencies will evaluate the recycled aggregate under the same standard specifications as their virgin aggregate but will classify the granular material as fill or sub-base (City of Edmonton 2009, City of Winnipeg 2010). As the recycled materials contain either asphalt cement or Portland cement they will have unique qualities that are outside the scope typically measured for a granular aggregate. In order to understand what additional qualities that the recycled aggregate has agencies need to understand where it is being used and how each recycled material type can provide additional benefits to the roadway structure.

As the City of Saskatoon had large stockpiles of asphalt rubble and portland cement concrete rubble generated from typical public works operations and other City projects, stockpiled rubble material was considered as a possible aggregate substitute. In 2006, the City of Saskatoon started using crushed portland cement concrete as a structural drainage and structural strain dissipation layer within a reconstructed roadway. With high contractor demands due to increased land development and high aggregate prices, the City further added to the recycled product use in 2007 by using City of Saskatoon forces to completely rebuild the granular structure of several structurally failed roadway sections using recycled asphalt and recycled portland cement concrete rubble aggregates. Figure 2.16 and Figure 2.17 show typical stockpiles of portland cement concrete and asphalt at one of the City of Saskatoon rubble reclamation yards.

Figure 2.16 Typical COS Portland Cement Concrete Rubble Stockpile

Figure 2.17 Typical COS Asphalt Rubble Stockpile

2.6.3 Historic Recycled Portland Cement Concrete Aggregate Material Use in Saskatoon

There are currently no specified uses for portland cement concrete rubble aggregate in the City of Saskatoon (City of Saskatoon, 2011). The only suitable use for recycled portland cement concrete according to the latest City of Saskatoon specifications is as a sub-base or pit-run aggregate. Sub-base and pit-run aggregates are considered the lowest quality aggregate available and is typically used as a filler granular when extremely poor subgrade is encountered in construction.

It is only since 2007 that the recycled portland cement concrete has been placed as a structural drainage aggregate (PSI Technologies Inc., 2010). The material that was used in the first projects was processed with a jaw and cone crusher and while it exhibited better drainage characteristics than base aggregate, had too many fines passing the 0.075mm sieve to fall out of the base gradation into the sub-base gradation. Further work with crushed portland cement concrete included more control on the gradation and therefore produced a more crushed rock type aggregate to be used as a drainage rock.

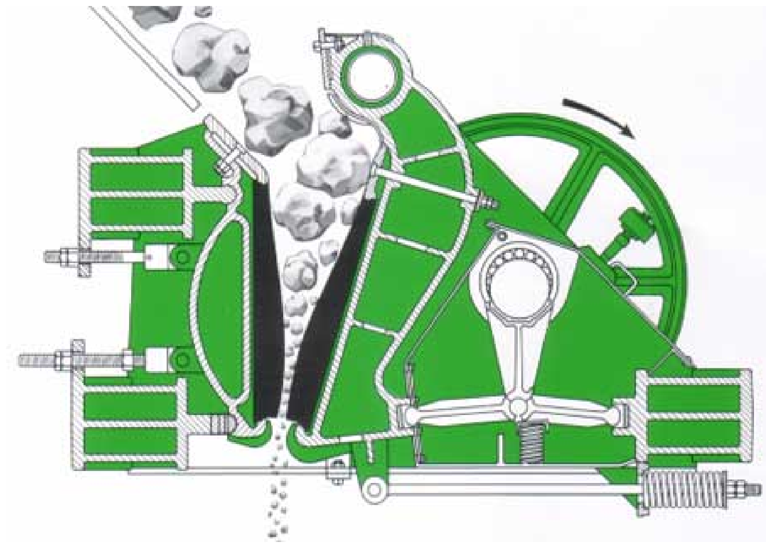
Portland cement concrete rubble from various demolition projects provides a high quality aggregate when crushed and has been the most commonly used recycled aggregate in the granular structure. Portland cement concrete rubble from building demolitions, concrete roadway resurfacing, sidewalk replacement and other applications has been processed into a sub-base graded aggregate (Berthelot et al. 2009). As part of the green street project, the rubble from a building demolition was stockpiled, as shown in Figure 2.18, and then processed into aggregate.

Processing building demolition waste has to be coupled with strict incoming material quality controls to provide a material worth processing. As of 2004 in the United States, 38 of the states have crushed portland cement concrete specified as an acceptable road sub-base material (Federal Highway Administration, 2004). This aggregate will typically range in size from 70μm up to75mm in diameter. The typical process used to crush this rubble is a jaw and cone type processor, shown in Figure 2.19, which grinds the material down to the required size.

Figure 2.18 Building Demolition Material Stockpiled at Dundonald Yard

Jaw and cone processed portland cement concrete type of material will have a gradation that passes the agency requirements for a sub-base type material. Sub-base is required to provide the load support as well as climatic capillary break between the subgrade for the granular base and pavement surface. Given the large particle size and the strength of the material it is well suited for this application. However, when processed with a significant portion of fines, material passing the 71 µm sieve, does lose some or all of the drainage capability as a granular material. As well, the process used in the jaw and cone crushing process tends to grind the aggregates as well as the portland cement concrete into either rounder aggregates or flat and elongated particle shapes rather than angular fractured materials that provide a stronger structural capacity.

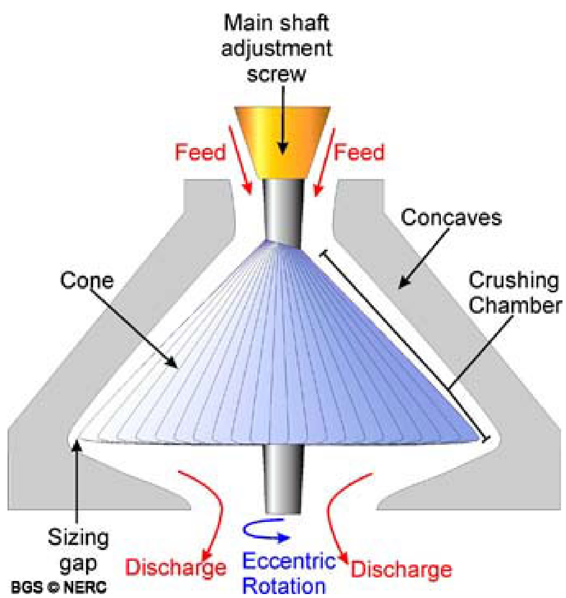
This fracture quality led the City of Saskatoon to investigate higher value uses for the recycled portland cement concrete. The Green Street project initiated in 2009 indicated that a high value aggregate useful for drainage applications could be produced with an impact crusher (Berthelot et al. 2010). Aggregate produced as shown in Figure 2.20 met crushed rock gradations and were shown to have high structural values (PSI Technologies Inc. 2010, Berthelot et al. 2010a, Berthelot, et al. 2010b).



a) Typical Jaw Crusher

(http://www.penncrusher.com/Size_Reduction/Models/Jaw_Crushers.cfm)

b)



b) Typical Cone Crusher (http://www.bgs.ac.uk/planning4minerals/Resources_21.htm)

Figure 2.19 Typical Jaw Crusher and Cone Crusher

Figure 2.20 Coarse Portland Cement Concrete Aggregate Produced in Green Street Project (PSI Technologies Inc., 2010)

2.6.4 Historic Recycled Portland Cement Concrete Aggregate Material Use in Other Agencies

The City of Edmonton operates a recycling yard where they process in excess of 200,000 tonnes of recycled aggregate per year (City of Edmonton, 2010). The rubble contains a mix of portland cement concrete, asphalt, and brick materials. City of Edmonton crushed rubble aggregate specification is to process the rubble material into a sub-base aggregate substitute and use that material as a road construction aggregate (City of Edmonton, 2009). While creating sub-base graded aggregate does recycle rubble, the specification does not use the rubble to its full potential as these rubble materials have significant structural capacity in their original application. This same process and application is used in the City of Regina as well within their own city forces work (Duce 2010). The City of Winnipeg specifications allow recycled portland cement concrete and asphalt to be used as a sub-base material, provided it meets certain gradation and mix characteristics. The province of Ontario allows recycled portland cement concrete to be substituted as a base aggregate as long as it meets all the physical and empirical testing measures required for virgin base aggregates (Government of Ontario, 2008).

Most states in the USA have adopted the use of recycled concrete as a base aggregate (Federal Highway Administration, 2004) with agencies such as the Texas department of transportation (TxDOT) recycling over 1 million tonnes of recycled concrete a year into the roadway. TxDOT has done work on evaluating the guidelines for the use and specification of recycled portland cement concrete and asphalt cement as aggregate in the roadways (Texas Department of Transportation, 2008).

Significant gains have been made in the last five years in reintroducing the recycled portland cement concrete back into new portland cement concrete applications. The portland cement concrete is processed and carefully sorted into specific sizes. This material is then used as a portion of the aggregate in new, non-structural portland cement concrete applications. However, this practice requires strict quality control on the incoming rubble and the processing gradations to provide a suitable aggregate (Gonzalez and Moo Young 2003, PSI Technologies Inc. 2010). Currently, recycled portland cement concrete material has not been approved in any portland cement concrete structures other than sidewalks, driveways and low volume roadways but is starting to be used in test sections throughout the United States and Canada.

2.6.5 Historic Recycled Asphalt Aggregate Material Use in City of Saskatoon

When urban agencies are faced with removing asphalt there are two typical methods employed. The first method is to remove the rubble with a backhoe or other bucket equipped machine as shown in Figure 2.21.

Through the removal process, the raw asphalt rubble material will typically have a mix of large asphalt pieces and other granular material mixed in. Depending on the care of the equipment operator, there may be other materials present as well with the most common contaminant being the *in situ* sub-grade. This method of asphalt concrete removal has produced rubble that, in the past, has been deemed unsuitable for anything other than general backfill. Often, in order to dispose of this material, it would have to go to the landfill or be buried somewhere off site. As there are environmental concerns with burying the asphalt material, there is a need to evaluate the best method of using the asphalt rubble to divert it from the landfills.

Figure 2.21 Typical Asphalt Removal Procedure

Figure 2.22 Typical Milling Operation in the City of Saskatoon

Typical jaw and cone crushing operations are unable to process asphalt rubble due to its higher levels of clay contamination as well as the cohesive characteristics of the asphalt rubble. Attempts to process this material in Saskatoon with these crushers have had limited to no success.

The second type of asphalt removal is with a cold milling machine that grinds up the top portion of the existing asphalt pavement surface and subsurface granular layer if required as shown in Figure 2.22. The asphalt is ground up into an aggregate which has been used as a replacement aggregate in new asphalt mixes. However, due to the high variability of the milling gradation, level of oxidation, and other physical unknowns there is significant uncertainty in the process of reusing the rubble in the asphalt mix. The City of Saskatoon does not currently allow any recycled asphalt in the new hot mix. Various trial mixes have been done in the past as modifications to the City of Saskatoon mixes allowing for up to 10 percent Reclaimed Asphalt Product (RAP) and the SMHI has had success with up to 30 per cent RAP. However, these mixes have not been used in typical City of Saskatoon construction.

There are two main uses of recycled asphalt in the City of Saskatoon. The primary use of the aggregate is as a base aggregate in back lanes and roadway utility cuts. The RAP has been the material of choice for these applications since 1995 due to the ability of the granular to seal and provide a better surface for traffic. In the back lanes, the RAP will provide better moisture resistance and a smoother surface than simply placing more base granular. As a temporary surface on the utility cuts, the RAP requires less maintenance to keep the surface drivable prior to being resurfaced.

The second use of the RAP is as a substitute base aggregate in roadway reconstructions as shown in Figure 2.23. This material has been used in that application since 2008 and has experienced some success. Little was known as to how the material would respond and how to specify the material for construction. Since it did not fit in with current specifications it was used on an experimental basis with City Public Works forces performing the construction. It worked well in this application and only had issues when the incoming asphalt aggregate was from a segment of the processed pile that had more fines than average.

Figure 2.23 RAP Used as Base Aggregate on Marquis Drive

Figure 2.24 Finished RAP Surface after Emulsion and Precipitation Event

When combined with two percent emulsion and compacted, the material forms a very strong platform for asphalt paving that has a low risk of moisture penetration. The finished product is shown in Figure 2.24.

2.6.6 Historic Recycled Asphalt Aggregate Material Use in Other Agencies

The City of Winnipeg allows the use of asphalt millings, on a case by case basis, as base aggregate based on a maximum particle size of 40mm. All drainage aggregate is specified as being uncoated which excludes all recycled portland cement concrete and asphalt materials. RAP may be used in specific asphalt mixes up to a maximum of 10 percent. The province of Ontario allows up to 30 percent RAP to be included in their base aggregates (Government of Ontario, 2008).

Reusing asphalt millings in new asphalt mixes is a common practice in Europe and North America. Many of the mixes in France are up to 80 percent recycled aggregate (Schimmoller, et al., 2000) and depending on the type of paving can be up to 100 percent in some states (Recycled Materials Resource Center, 2008). It is standard for recycled aggregates to be tested in the same manner as standard aggregates and the recycled aggregates are required to have at least equal mechanical qualities for it to be considered as a substitute for the virgin aggregate (PIARC Technical Committee C4.3 Road Pavements, 2008). The areas with the most prevalent use of recycled asphalt are typically in warmer climates. With increased recycling comes stiffer mixes that may be susceptible to climatic cracking.

The main aggregate lab test in North America historically used was the California Bearing Ratio (CBR) Test. However, many agencies have switched over to evaluating the resilient modulus (M_R) in order to evaluate the aggregates and subgrades of the roadway structure. Recycled asphalt typically will have higher resilient modulus values than conventional granular aggregates (Kim & Labuz, 2007). Correlation equations are then adopted to relate the CBR and the M_R in order to make use of empirical data. Typically, recycled asphalt has been avoided as a base aggregate in road building due to its reduced CBR from the typical granular structure (Guthrie, Cooley, & Eggett, 2007). As well, resilient modulus testing of recycled asphalt has shown an increase in permanent deformation in RAP bases when compared to conventional base materials (Kim & Labuz, 2007). However, test section evaluations have been performed and have found that the recycled asphalt has better structural response than granular even though the CBR values indicated a poorer structural measure (Sayed et al 2011). This discrepancy between CBR and field performance is likely due to the effect of the cohesive state provided by the residual asphalt and hydrophobic nature of asphaltic millings.

2.7 Chapter Summary

The City of Saskatoon is growing by 2.3 percent per year. The growth corresponds with an additional 300,000 square meters of new roadways constructed each year costing over 4.5 percent of the entire City of Saskatoon operating and capital budget. Current City of Saskatoon design methodology does not address the issues of poor subgrades, critical state loading and depleting aggregate resources. Therefore, a significant financial liability has been identified in the current City of Saskatoon roadway design process.

One alternative option for aggregate resources is the use of recycled portland cement concrete and asphalt aggregates. In order to incorporate recycled aggregates the specifications need to be revised to allow the use of recycled aggregates and the design process needs modification to account for the material characteristics.

As the design system currently used by the City of Saskatoon does not account for critical state loading, the design process needs to be modified to account for the critical state loading, more testing done to properly characterize the subgrade material and design much thicker structures adequate for traffic loading greater than 10^5 ESALs in the SMHI modified shell curve nomographs.

Designing urban roadways with drainage structures will satisfy the design assumption of a high dry granular structure used by the City of Saskatoon. These drainage structures are currently being constructed by the City of Saskatoon using crushed rock and crushed portland cement concrete as the drainage aggregate. However, there are no specifications in the City of Saskatoon standard specifications for this application or means of designing the structures with the drainage layer.

CHAPTER 3 CONVENTIONAL GRANULAR MATERIAL CHARACTERIZATION

In order to properly specify recycled materials, a method for comparison against conventional road building aggregates needs to be employed. The current specifications rely on empirical correlations of field performance of known virgin aggregates but current specifications may not correlate well with recycled aggregates. The characterization conducted in this study will evaluate some of the empirical testing that is currently specified for road aggregates as well as some of the mechanistic testing to evaluate aggregates from first principals.

Conventional aggregate testing includes the full spectrum of physical characterization. In this traditional protocol aggregates are identified by aspects such as grain size characterization, number of fracture faces, plasticity of fines, various soil classification tables and a test known as the California Bearing Ratio.

3.1 Grain Size Distribution (ASTM C136 and ASTM C117)

The sieve analysis performed by test method ASTM C136 classifies the amount of granular material by weight across various sizes. ASTM C117 classifies the material finer than 75µm into various sieve sizes by washing as illustrated in Figure 3.1. Jurisdictions then use the fractions identified in each size to specify different material types. As shown in Chapter 2, the City of Saskatoon identifies different gradations required for the various aggregates. In order to incorporate the recycled aggregates into the City of Saskatoon specifications it is important to know what gradations of recycled aggregates are possible.

It is possible to process recycled aggregates into gradations equivalent to conventional granular materials. The green street evaluation process started in 2009 by the City of Saskatoon was conducted to evaluate what materials could be produced from portland cement concrete rubble and asphalt rubble. In all, five separate gradations of processed portland cement concrete were produced and three grades of processed asphalt were produced.

Figure 3.1 Wet Sieving of Granular Aggregate (ASTM C117)

The first pilot project of portland cement concrete rubble processing evaluated the process into two gradations attempting to maximize the amount of crushed rock graded material. This pilot project produced a secondary material that was higher in fines than specifications for base allowed. Evaluation and screening expanded to produce more material types. This allowed second generation processing to retain more valuable material in addition to creating aggregates that met two of the gradations fall within City of Saskatoon specifications as summarized in Table 3.1 and illustrated in by Figure 3.2 (PSI Technologies Inc., 2010). The 19 mm Open Graded Base Course (OGBC) was an additional material produced within the Green Street Evaluation. Although the OGBC was not within any of the specified gradations, the OGBC was determined to be a high quality drainage aggregate due to its high fracture count and relatively uniform size resulting in a high permeability as well as high internal coefficient of friction for structural integrity.

Table 3.1 Processed Portland Cement Concrete Gradations

Diameter (mm)	19mm PCC (GW)	19mm PCC (OGBC)	25mm PCC (OGBC)
50	100	100	100
25	100	97.8	20.8
18	90.2	89.6	5.1
12.5	81.4	81.6	4.6
5	57.3	37.3	4.3
2	40.5	9.6	4
0.9	30.5	5.3	3.6
0.4	22.5	4.2	3.2
0.16	15.6	3.3	2.6
0.071	9.9	2.5	1.7

Following the protocols learned from processing portland cement concrete rubble, the asphalt rubble was also processed into multiple gradations (PSI Technologies Inc., 2010). Two of the gradations produced are summarized in Table 3.2 and illustrated in Figure 3.3. As shown, they were able to be produced within the gradations required for crushed rock and base course for the City of Saskatoon. In addition to these materials, a large coarse asphalt rock was produced to minimize the amount of aggregate breakdown within the asphalt rubble.

In processing the rubble material into the various specified gradations it was found that very little to no waste material was produced.

Table 3.2 Processed Recycled Asphalt Gradations (PSI Technologies Inc. 2010)

Diameter (mm)	19mm RAP	25mm RAP
25.0	100.0	39.0
18.0	96.2	11.6
12.5	85.2	8.6
5.0	53.0	6.4
2.0	33.5	5.5
0.9	23.4	4.6
0.4	16.5	3.8
0.16	10.8	3.0
0.071	8.2	2.5

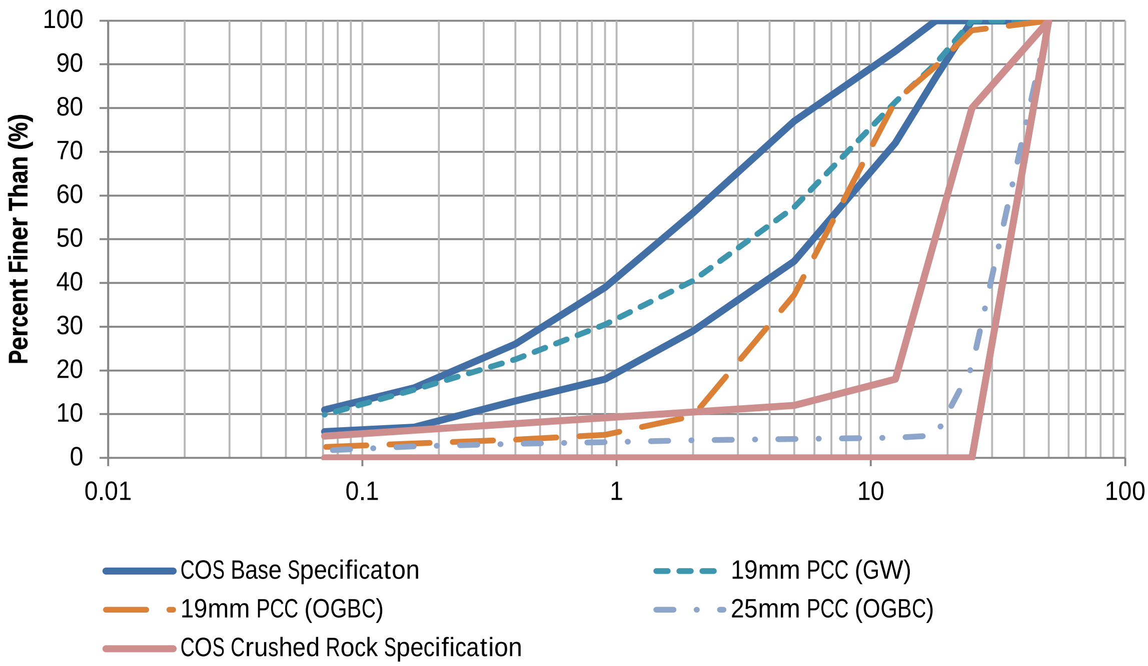


Figure 3.2 Processed Portland Cement Concrete Gradations (PSI Technologies Inc. 2010)

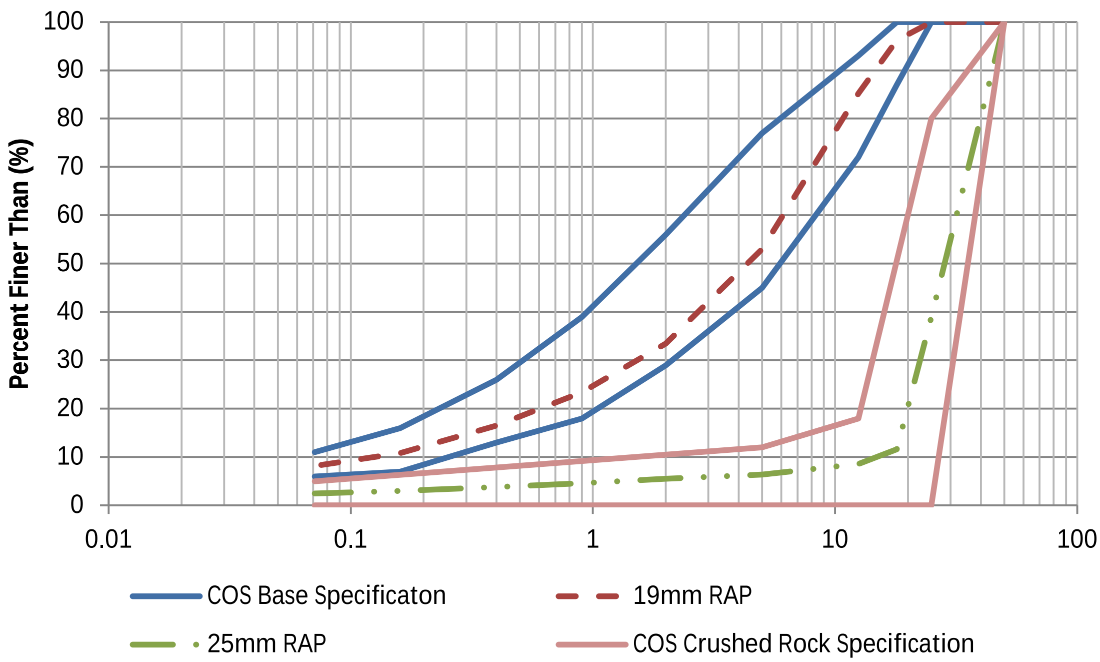


Figure 3.3 Processed Recycled Asphalt Gradations (PSI Technologies Inc. 2010)

3.2 Aggregate Classification (ASTM D2487 and ASTM D3282)

When comparing the gradations shown in Section 3.1.1, the recycled aggregates would be classified as Class A-1-a aggregates in the AASHTO system.

With the USCS classification identified in Table 2.3, the base graded crushed portland cement concrete would be a silty gravel (GM) or clayey gravel (GC) as the fines are greater than 12 percent. Again, as a high percentage of the fines would be cementitious, these classifications do not fit well. The other crushed portland cement concrete gradations are classified as well graded gravel (GW). When classifying the crushed asphalt, the base graded crushed asphalt would be GW while the others would be classified as poorly graded gravel (GP).

3.3 Aggregate Angularity (ASTM D5821)

Aggregate comes in different shapes as well due to the type of processing and source location. When used in construction, the amount of fractured faces, or angularity, is important in determining its suitability for different applications. Figure 3.4 illustrates material being separated to determine angularity.

The angularity test takes a sample of the aggregate material and separates the rocks into fractured and non-fractured piles. The number of rocks in each pile is then counted and a ratio of angularity is determined. The greater the amount of angularity, the more effort that is required to compact and the greater the load bearing capacity of the material due to internal friction and aggregate interlock. ASTM D5821 defines the process used to determine the aggregate angularity of the fine aggregate and course aggregate. Table 3.3 and Figure 3.5 illustrate the aggregate angularity of the test materials identified. All recycled aggregates were found to exceed the required particle angularity for the COS specifications of 50 percent.

Figure 3.4 Aggregate Angularity Test

Table 3.3 Angularity of Recycled Aggregates

Recycled Aggregate		**Fracture (% on material >5mm)**
Conventional Base	19mm (GW)	49
RAP	19mm GW	53
	25mm OGBC	72
	CIP RAP	67
PCC	19mm (GW)	96
	19mm (OGBC)	89

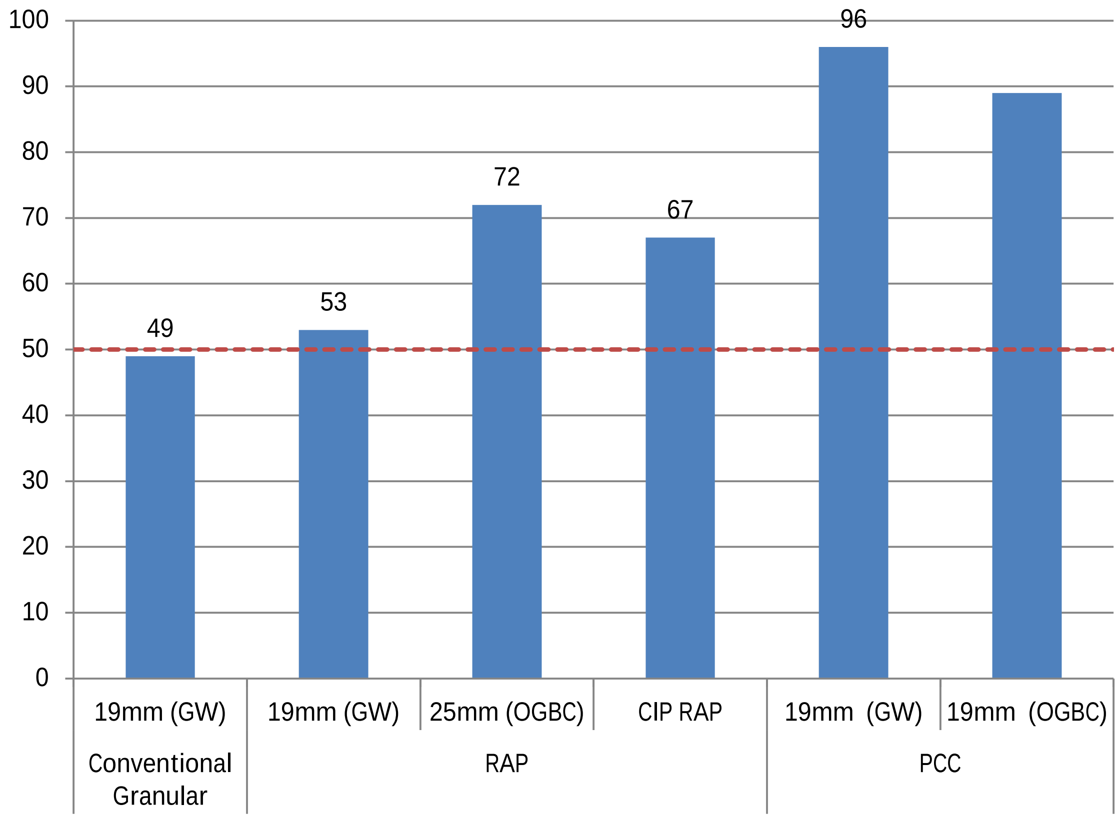


Figure 3.5 Aggregate Angularity Results

3.4 Standard Proctor Density (ASTM D698)

The standard proctor density test was created in order to determine the amount of compaction required on a soil. This test is performed by compacting soil samples a set amount across a number of moisture contents. The dry density of each type of compacted sample is then calculated and plotted versus the moisture content. The peak of this plot is determined to be the maximum density and optimum moisture content of the soil type. As outlined in Table 3.4 and illustrated in Figure 3.6, each of the tested samples had variations in their optimum moisture content and maximum dry density.

Table 3.4 Moisture Density Optimums of Granular Materials

Material	Optimum Moisture Content	Dry Density (kg/m^3)
19mm Conventional Base (GW)	6.5%	2220
19mm High Fracture Base (GW)	5.0%	2260
19mm RAP (GW)	3.2%	2170
25mm RAP (OGBC)	1.9%	2200
CIP RAP	5.7%	2029
19mm PCC (GW)	6.0%	2002
19mm PCC (OGBC)	5.2%	1876

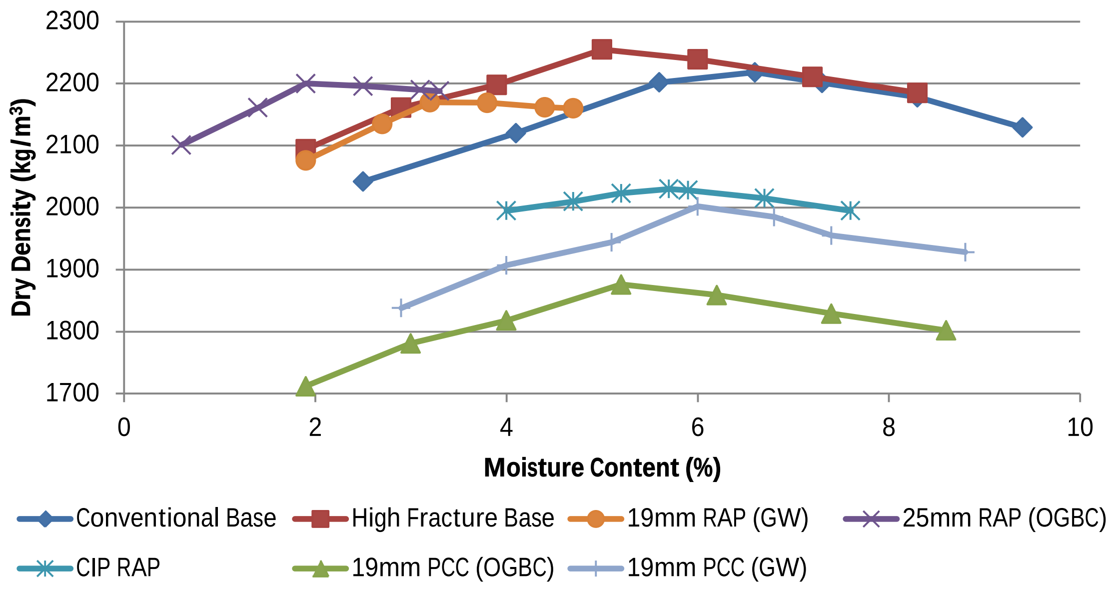


Figure 3.6 Moisture Density Curves of Granular Aggregates

3.5 California Bearing Ratio (ASTM D1883)

The California Bearing Ratio (CBR) is a test method that has been used since 1961 in road construction to compare the relative load bearing capacity of the soil and granular materials (ASTM D1883, 2007). The standardized test compares how much effort is required to push a 50mm diameter rod a measured depth into the soil. Figure 3.7 illustrates how the apparatus used to perform this test.

Figure 3.7 Standard CBR Testing Apparatus

The ratio of the measured effort in relation to the standard effort required for California crushed limestone is termed the California Bearing Ratio.

$$CBR = \frac{\rho_m}{\rho_{st}} \times 100 \qquad [3.1]$$

Where:

ρ_m = pressure required in medium

ρ_{st} = pressure required in the standard crushed limestone

Soils can either be evaluated in a soaked or unsoaked condition to evaluate how they will react in the field. The highways design manual calls for an unsoaked CBR to be taken on the granular due to the assumption of a high and dry structure. As the City of Saskatoon specifications are based on the highway design manual, the unsoaked CBR test is also used. City of Saskatoon specifications require an unsoaked CBR of at least 65 percent (City of Saskatoon, 2011). As shown in Table 3.5 and illustrated in Figure 3.8, the recycled materials do not meet this standard under the conventional sample preparation testing method. The ASTM standard for CBR specifically states that the test is only meant for subgrades and unbound aggregates. As the recycled aggregates are bound by residual asphalt and cement, the CBR test is not an accurate measure of their qualities.

Table 3.5 Unsoaked CBR Test Results - Proctor Compacted

Granular Material	CBR Peak Strength (%)
19mm Conventional Granular (GW)	77
19mm RAP (GW)	14
25mm RAP (OGBC)	3
CIP RAP	19
19mm PCC (GW)	41
19mm PCC (OGBC)	22

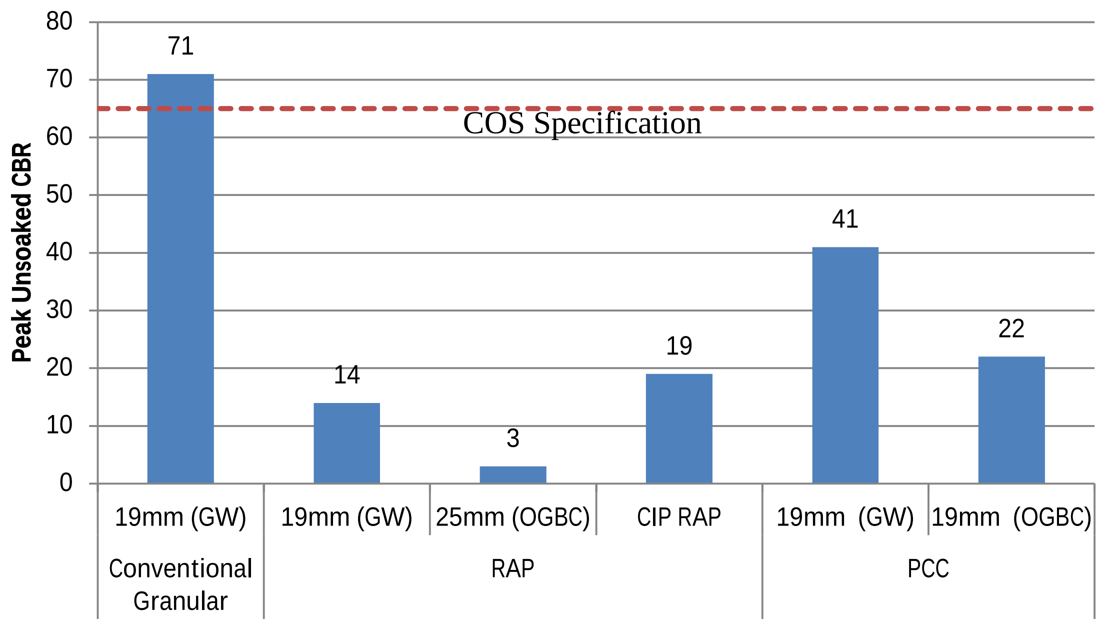


Figure 3.8 Unsoaked CBR of Recycled Aggregates

However, the information gathered in the soaked CBR test facilitates the understanding about the granular material behaviour in "wetted up" field state conditions. Road designers are interested to know how the material will swell in the presence of water since this will significantly reduce the strength of the granular material and more accurately represents the field state conditions in the road failure locations. As well, modifying the CBR compaction method to gyratory compaction was used to allow simulative of field compaction of the recycled granular aggregates. Gyratory compaction was found to be required since the recycled aggregates contained asphalt cement and Portland cement as a stabilizing agent. Table 3.6 and Figure 3.9 indicate the CBR soaked peak strength of the various crushed aggregates when compacted using gyratory compaction. The OGBC RAP and both PCC aggregates performed over 60 percent better than conventional granular due to the high fracture content.

Table 3.6 Soaked CBR Test Results – Gyratory Compacted

Granular Material	CBR Peak Strength (%)
19mm Conventional Granular (GW)	75
19mm RAP (GW)	60
25mm RAP (OGBC)	122
CIP RAP	14
19mm PCC (GW)	123
19mm PCC (OGBC)	134

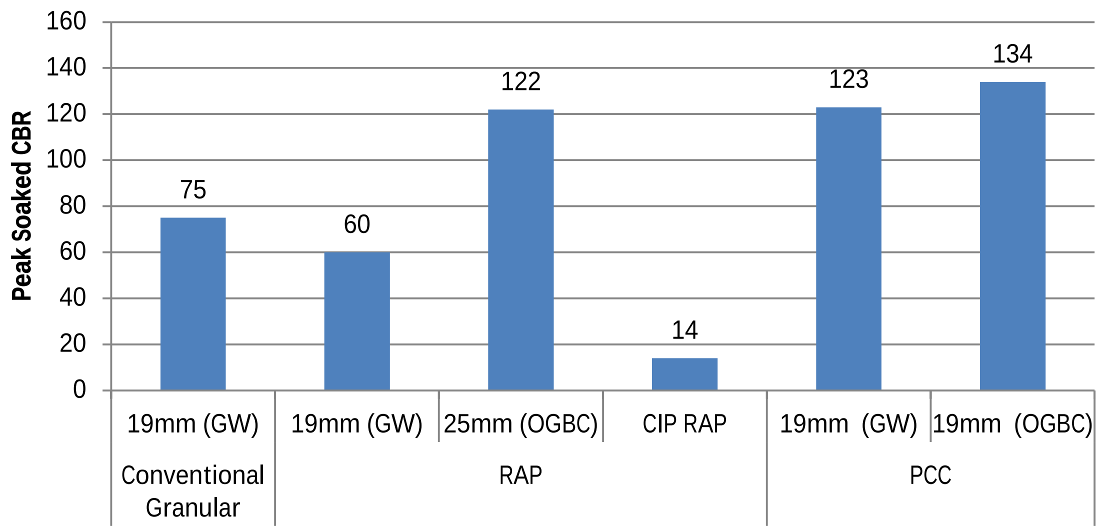


Figure 3.9 CBR Peak Strength For Test Materials

The recycled aggregate materials react differently to CBR testing if compacted with a Proctor hammer or through the gyratory compactor. As the gyratory compactor more realistically models how granular material is compacted in the field, this method was used to determine the CBR of the recycled granular aggregates.

When looking at the soaked swell of the materials indicated in Table 3.7 and illustrated in Figure 3.10, all recycled materials had less swelling behaviour when soaked than conventional base. This swell behavior indicates that when the recycled materials do take on some water they do not swell and therefore are less susceptible to moisture damage compared to typical granular base.

Table 3.7 CBR Soaked Swell Results across Material Type

Material	CBR 96-Hour Soaked Swell (%)
19mm Conventional Granular (GW)	0.73
19mm RAP (GW)	0.33
19mm RAP (OGBC)	0.15
CIP RAP	0.13
19mm PCC (GW)	0.28
19mm PCC (OGBC)	0.01

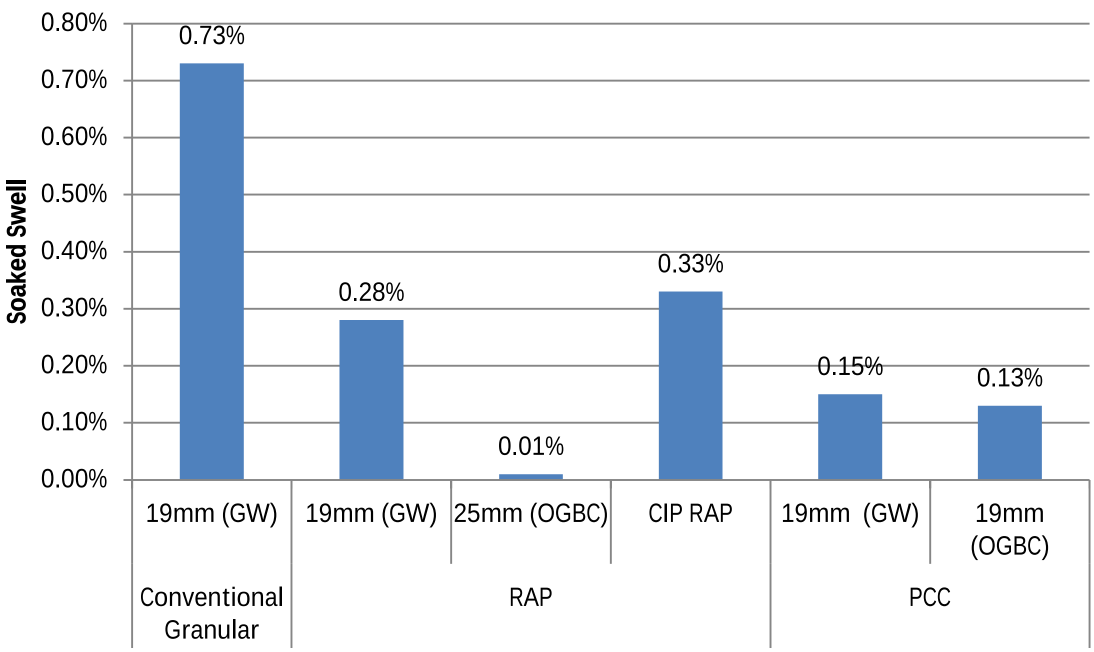


Figure 3.10 CBR Soaked Swell Results across Material Type

3.6 Atterberg Limits and Plasticity Index (ASTM 4318)

When classifying soils, the plastic index is a common tool used to determine the type of fines within the material and how it responds to moisture. The index is the relationship between how much moisture is required for the soil to reach its plastic limit as well as its liquid limit.

When Atterberg limit tests were attempted on the recycled asphalt and portland cement concrete aggregates, the samples exhibited properties that did not allow limit measurements to be taken. Therefore, it was assumed that all granular aggregates exhibited a plastic index of 0 and are classified as non plastic which meets the COS requirements for sub-base which requires a plastic index of less than 6.0..

3.7 Sand Equivalency (ASTM D2419)

The sand equivalency test is performed to determine the type of fines present in the aggregate. The City of Saskatoon does not specify a minimum sand equivalency but the test is used in other jurisdictions. The portion of the aggregate passing the 0.4 mm sieve is placed in a cylinder with water and a flocculent to separate the clay fines from the sand aggregates as shown in Figure 3.11.

Figure 3.11 Sand Equivalency Testing Apparatus

The test measures the different levels of the particles to determine an equivalency ratio. A high sand equivalency indicates that the fines are mostly larger “sand-like” particles while a low sand equivalency indicates the presence of smaller sized plastic fines within the aggregate. Plastic fines are not desirable within the aggregate as the plastic fines attract moisture. When tested, the portland cement concrete fines react with the flocculent as well and remain suspended providing a false measure with the test methodology. The sample sand equivalent of 45 is lower than expected for most roadway applications.

3.8 Chapter Summary

This chapter shows recycled aggregates can meet and exceed the physical specified requirements for angularity and can be processed to meet gradation requirements set out in the City of Saskatoon specifications. In order to minimize the processing of the large aggregate, an open graded base course material was also processed and evaluated for the City of Saskatoon. However, the open graded material is not accounted for in the City of Saskatoon specifications.

Conventional tests had mixed results for recycled materials. All recycled aggregates met the minimum 50 percent fracture required by the City of Saskatoon. The CBR test done with a proctor compactor indicated that the recycled materials failed to meet minimum requirements. However, the same material compacted with a gyratory compactor and tested in the CBR, the gyratory compacted samples exceeded the minimum requirements for unsoaked aggregates even when evaluated in the soaked test.

The swell tests indicated that the recycled material was significantly less susceptible to moisture damage in freeze thaw cycles in relation to the conventional granular base.

The recycled aggregates all had a plasticity index of zero and would meet those standards for City of Saskatoon sub-base. Sand equivalent values were lower than expected for the recycled aggregate but are not tested for in standard City of Saskatoon specifications.

CHAPTER 4 MECHANISTIC TRIAXIAL FREQUENCY SWEEP CHARACTERIZATION

Road granular materials are used in high deviatoric and highly dynamic loading field state conditions. It is therefore imperative that road materials be evaluated in a similar framework in order to determine the roadway structural capacity. One of the tools used to evaluate the materials in this manner is the Rapid Triaxial Testing (RaTT) Apparatus shown in Figure 4.1.

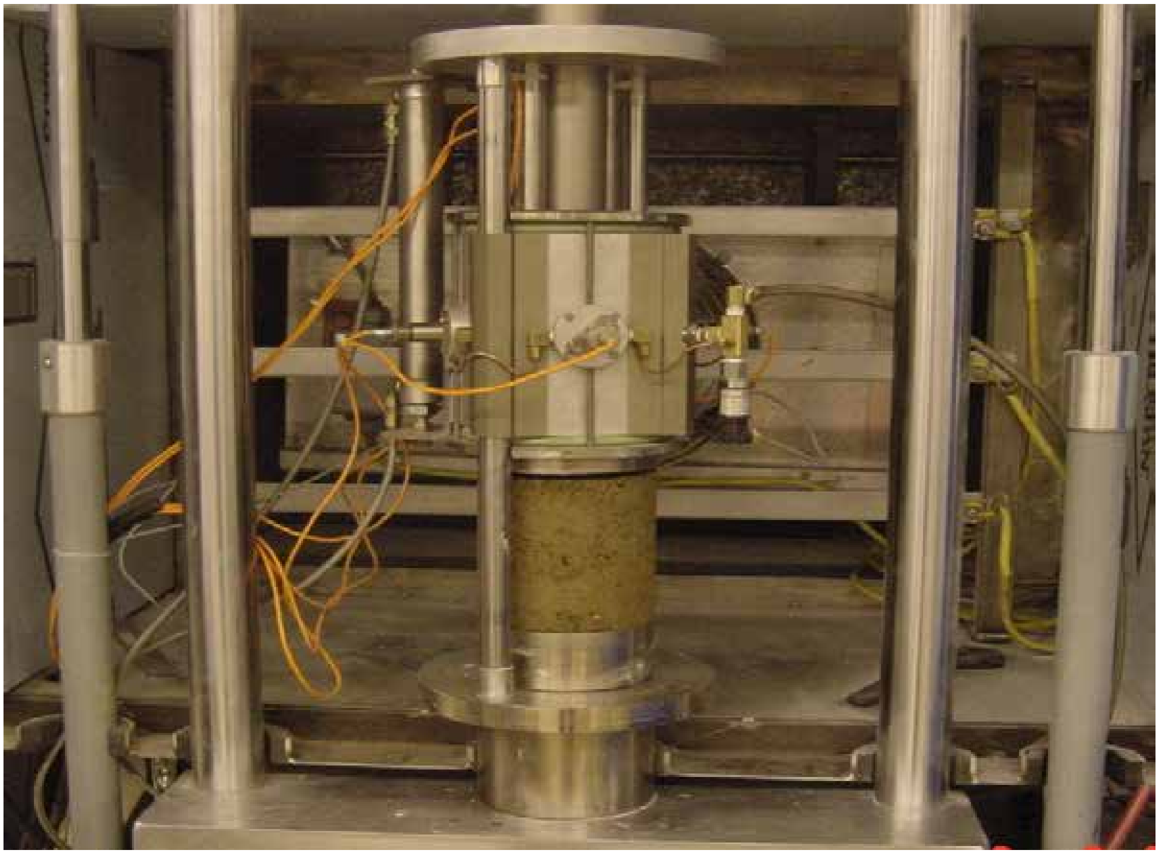


Figure 4.1 Rapid Triaxial Testing Apparatus

This testing apparatus has the ability to cyclically apply variable loads in all three axes. These variable loads adds the ability to change the stress states of the testing sample from a low stress state with low confining pressure to high stress state through to a full reversed stress state as illustrated in Table 4.1 where the confining pressure is greater than the vertical loading.

These different stress states simulate the material stresses through the different levels of the structure. Through an iterative process within the modelling performed in Chapter 5 the various stress states in the different materials with respect to depth in the road structure was calculated. The materials were then tested at these stress states to simulate the loading they would experience in their layer of the design structures. Figure 4.2 shows how the relative stresses, both from traffic loading and internal body forces, were used at the various depths.

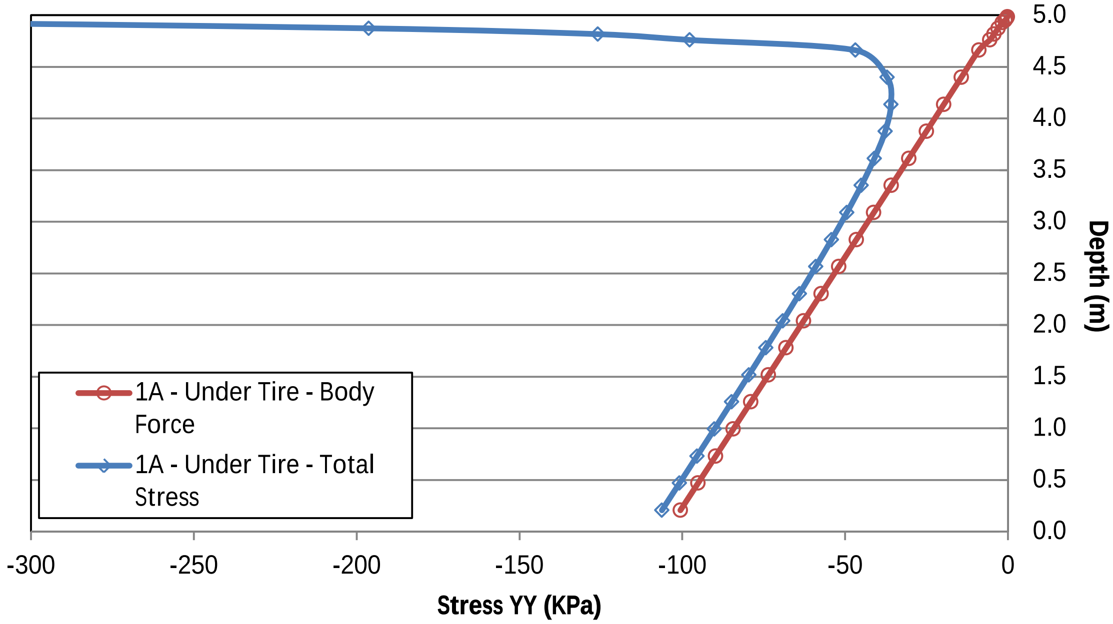


Figure 4.2 Stress State Loading Versus Depth

The stress state that the various aggregates in this study were tested under was the fully reversed stress state at four different frequencies. The fully reversed stress state illustrated in Table 4.1 shows the confining stresses being greater than the vertical compressive force. The increase in confining stress is to simulate the stress the aggregates experience near the surface but not directly under the load. The four frequencies are to simulate the velocity of traffic experienced in urban settings ranging from fast moving to slow moving traffic.

Table 4.1 Triaxial Frequency Sweep Stress State Testing Parameters

Stress State	Hydrostatic Stress	Deviatoric Stress
Fully Reversed	250 kPa	±200 kPa

4.1 Dynamic Modulus

Using the rapid triaxial testing apparatus the aggregates were tested under the field state conditions they would experience as determined by modeled information and knowledge of development areas. Conventional base aggregate failed when tested at the field state stresses. Therefore a high fracture base was tested as a comparison. Figure 4.3 and Table 4.2 indicate the range of dynamic modulus of the test materials over the four test frequencies.

The RAP material had at least 200 percent greater stiffness than the high fracture base when evaluated across frequencies. The stiffness benefit was significantly larger at high frequencies due to the asphalt content in the RAP. The crushed PCC did not vary over frequency although testing indicated the PCC had 80 to 100 percent higher stiffness than conventional base.

Table 4.2 Dynamic Modulus of Test Materials across Material Type and Load Frequency

	Dynamic Modulus (MPa)			
	10Hz	**5Hz**	**1Hz**	**0.5Hz**
High Fracture Base	251	254	259	264
19mm RAP (GW)	811	747	603	560
25mm RAP (OGBC)	1386	1185	813	697
CIP RAP	488	451	377	358
19mm Crushed PCC (GW)	408	402	385	385
19mm Crushed PCC (OGBC)	485	489	493	501

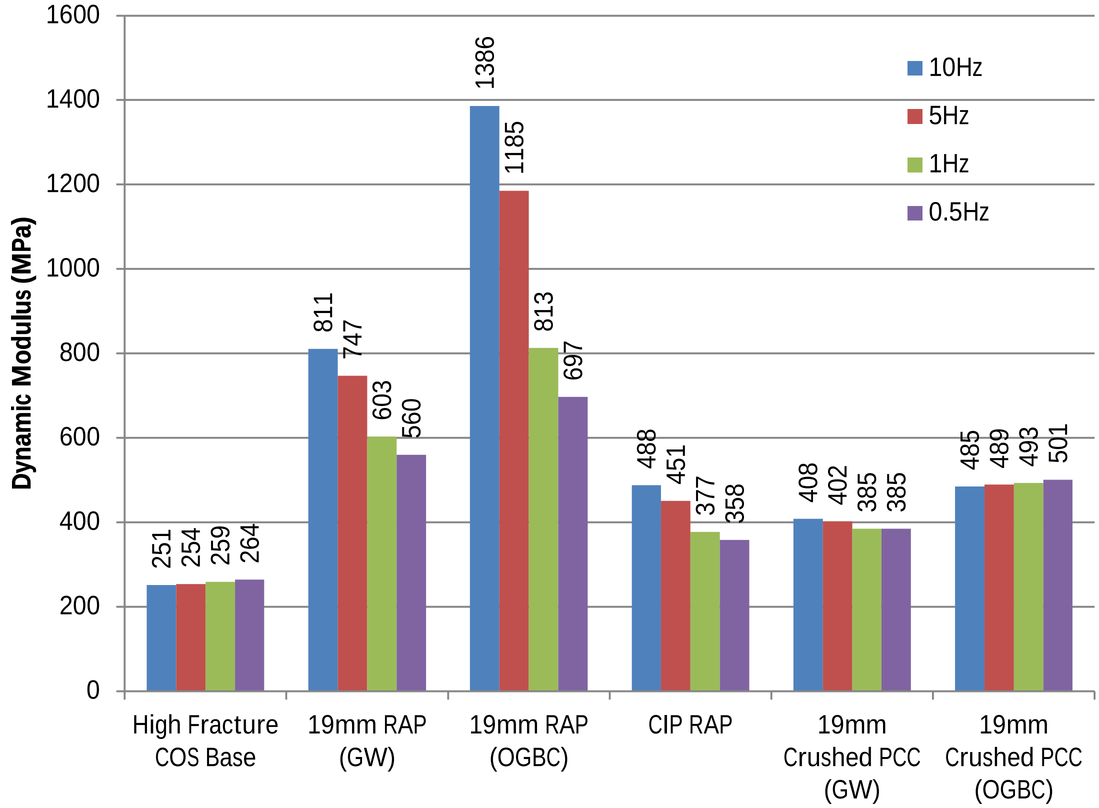


Figure 4.3 Dynamic Modulus across Test Materials across Material Type and Load Frequency

4.2 Poisson's Ratio

The various Poisson's Ratio shown in Figure 4.4 and Table 4.3 show how the aggregate materials will deform as they are loaded in their design field state conditions across a range of frequencies. The conventional base aggregate failed in the design field state conditions. Therefore, a high fracture base was evaluated in order to have a comparison available. The lower values shown in the recycled aggregates indicate better structural mechanical characteristics.

Table 4.3 Poisson's Ratio of Test Materials across Material Type and Load Frequency

	Poisson's Ratio			
	10Hz	**5Hz**	**1Hz**	**0.5Hz**
High Fracture Base	0.43	0.41	0.40	0.40
19mm RAP (GW)	0.31	0.32	0.33	0.33
25mm RAP (OGBC)	0.35	0.35	0.39	0.39
CIP RAP	0.37	0.37	0.37	0.37
19mm Crushed PCC (GW)	0.40	0.40	0.39	0.38
19mm Crushed PCC (OGBC)	0.31	0.31	0.31	0.31

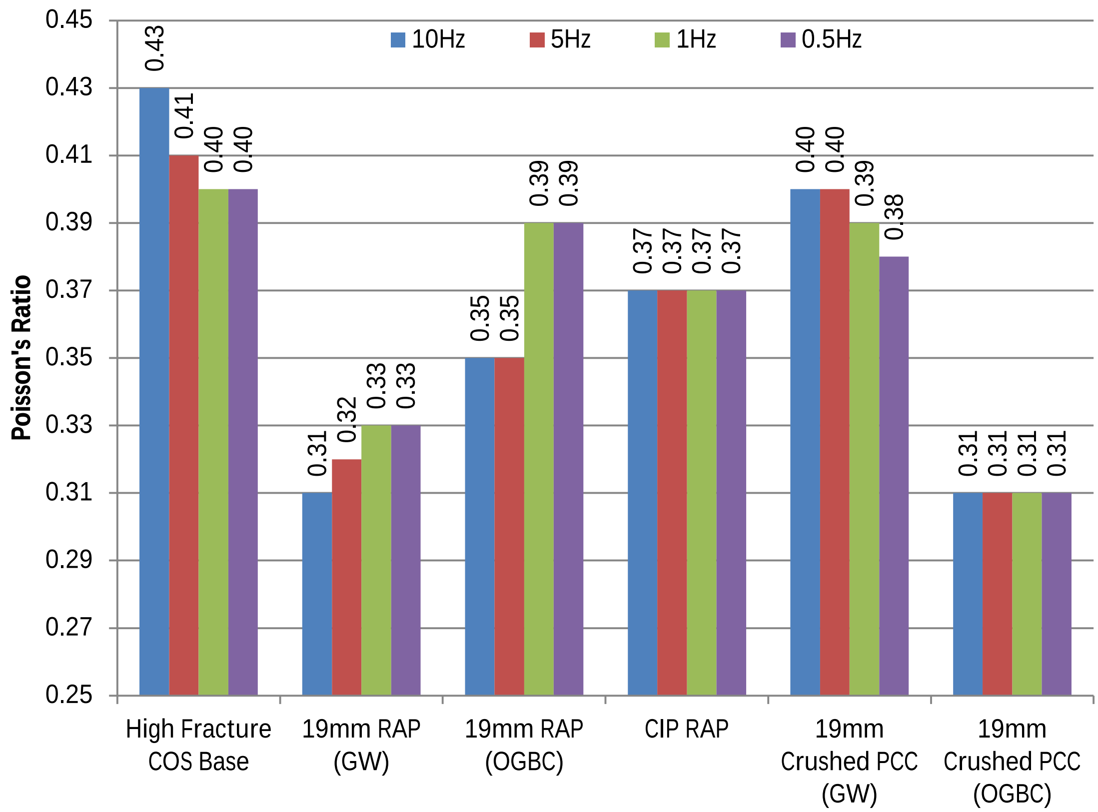


Figure 4.4 Poisson's Ratio of Test Materials across Material Type and Load Frequency

4.3 Phase Angle

The phase angle measures the lag between the stress and the strain. Higher phase angle values indicate the material acts viscously while lower values are indicative of elastic properties within the material. As shown in Table 4.4 and illustrated in Figure 4.5, the crushed PCC aggregate behaves more elastically than the other aggregates. The RAP and PCC have less variability with regard to frequency than the high fracture COS base.

Table 4.4 Phase Angle of Test Materials across Material Type and Load Frequency

	Phase Angle			
	10Hz	**5Hz**	**1Hz**	**0.5Hz**
High Fracture COS Base	18.1	15.7	12.7	11.8
19mm RAP (GW)	19.8	18.6	17.8	17.5
19mm RAP (OGBC)	18.0	17.1	17.1	17.3
CIP RAP	19.4	18.4	17.4	17.0
19mm Crushed PCC (GW)	12.2	10.8	9.4	9.3
19mm Crushed PCC (OGBC)	10.7	9.4	8.1	7.6

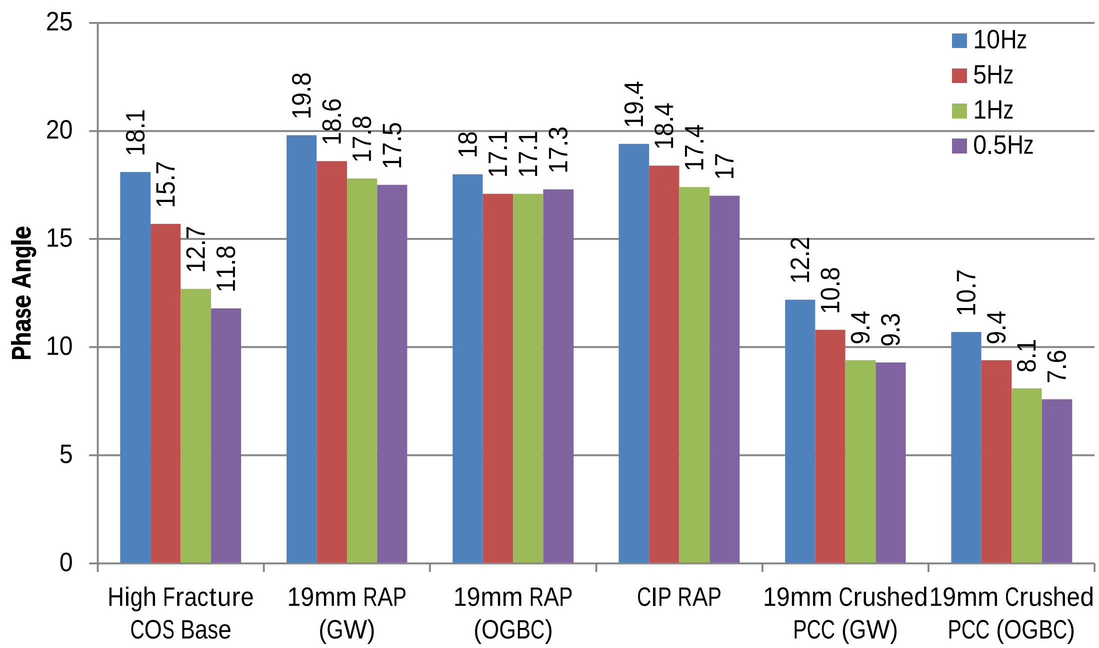


Figure 4.5 Phase Angle of Test Materials across Material Type and Load Frequency

4.4 Radial Microstrain

Within the Poisson's Ratio, the lateral strain is also referred to as radial microstrain. This characteristic can be used to evaluate the propensity of the material to fail in lateral shear. As shown in Table 4.5 and illustrated in Figure 4.6 the recycled material has a lower radial microstrain value than the high fracture COS granular base. The OGBC recycled aggregates, both asphalt cement and portland cement concrete, had the lowest radial microstrain, indicating that this gradation reduces the risk of lateral shear failure.

Table 4.5 Radial Microstrain of Test Materials across Material Type and Load Frequency

	Radial Microstrain			
	10Hz	**5Hz**	**1Hz**	**0.5Hz**
High Fracture COS Base	682	642	616	598
19mm RAP (GW)	239	263	318	341
19mm RAP (OGBC)	125	138	187	214
CIP RAP	298	324	393	412
19mm Crushed PCC (GW)	453	465	467	456
19mm Crushed PCC (OGBC)	256	250	247	242

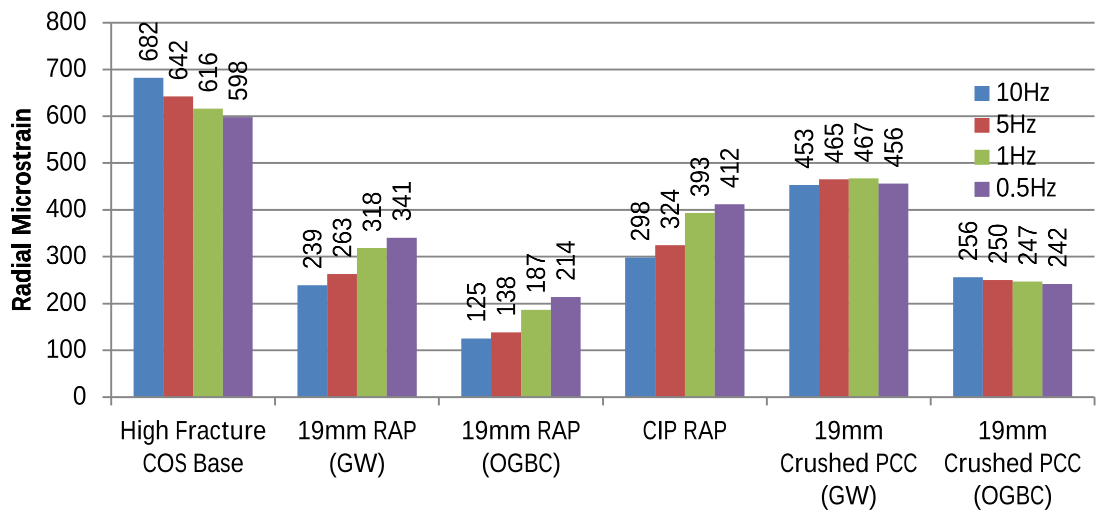


Figure 4.6 Radial Microstrain of Test Materials across Material Type and Load Frequency

4.5 Complex Shear Modulus

When comparing the various recycled aggregates to high fracture COS base, the recycled aggregates showed more resistance to stress than the high fracture base. The high values shown in Table 4.6 and illustrated in Figure 4.7 indicate a 38 percent to 585 percent increase in resistance to shear in the recycled aggregates. The OGBC RAP had the highest resistance to shear while the GW PCC had the lowest of the recycled aggregates.

Table 4.6 Complex Shear Modulus of Test Materials across Material Type and Load Frequency

	Complex Shear Modulus			
	10Hz	**5Hz**	**1Hz**	**0.5Hz**
High Fracture COS Base	87.8	90.1	92.5	94.3
19mm RAP (GW)	309.5	283.0	226.7	210.5
19mm RAP (OGBC)	513.3	438.9	292.4	250.7
CIP RAP	178.1	164.6	137.6	130.7
19mm Crushed PCC (GW)	145.7	143.6	138.5	139.5
19mm Crushed PCC (OGBC)	185.1	186.6	188.2	191.2

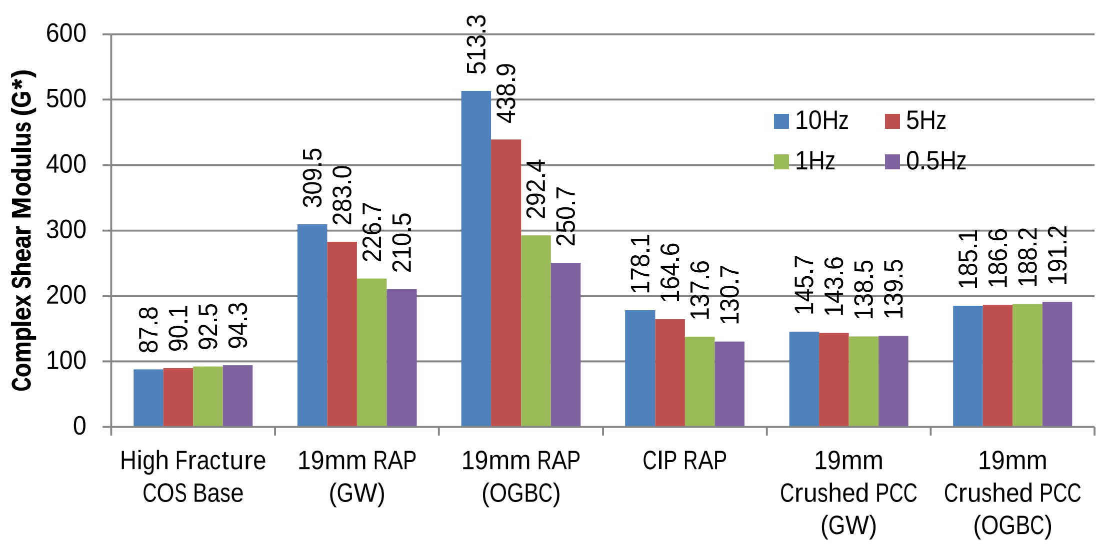


Figure 4.7 Complex Shear Modulus of Test Materials across Material Type and Load Frequency

4.6 Climatic Durability

There are two key climatic effects that need to be accounted for in any roadway design in Saskatoon's climate. The first climatic effect is the freeze thaw cycles that the City of Saskatoon experiences. There are three common methodologies used to minimize the damage caused by freeze thaw. The methodologies include adding insulation so that the soil does not freeze, choosing soil that is not moisture susceptible or minimizing the amount of water that can enter the system. As most of the City of Saskatoon roadways are in an urban environment, the insulation is not an effective solution due to all the utilities under urban roadways. Therefore, the City of Saskatoon needs to deal with the soil characteristics and the water availability. As the soil characteristics are also not usually able to change since most construction is done with *in situ* material, other than by adding stabilizing agents, dealing with the water availability is usually the solution chosen. A roadway design that keeps the granular structure dry will not require choosing granular material based on moisture susceptibility.

The samples underwent a suction and dielectric measurement test developed in Finland to evaluate the moisture susceptibility of different aggregates (Scullion & Saarenketo, 1997). The samples were placed in a small amount of water and the amount of capillary rise of moisture was measured until it stabilized. A high and quick intake of moisture indicated a higher risk of moisture susceptibility. As well, the dielectric value of the material measured after 24 hours is a means to measure the bonding of the water with the fine aggregates in the mix.

4.6.1 Moisture Intake

With the construction of roadways in the City of Saskatoon creating "clay boxes" around the roadway structure, the presence of moisture is an issue in the roadway design. Evaluating the various aggregates to determine how much moisture they will draw into the structure is important to quantify whether they are appropriate for roadway construction in urban designs. As each aggregate was tested to determine how much moisture intake occurred, it was found that the processed RAP and OGBC PCC were more appropriate to reduce moisture intake, while the well graded PCC was more susceptible to retaining moisture and therefore being at risk in the freeze thaw cycles. The results from the 24 hour moisture intake test are shown below in Table 4.7 and illustrated in Figure 4.8.

RAP mixtures contain asphalt as a stabilizer. Asphalt is typically hydrophobic and the RAP within the mix will not retain the moisture as much as a conventional aggregate once the source of the water is removed.

Cement fines as a binder within an aggregate, particularly a well graded aggregate will act similar to natural clay fines when dry. However, as the fines wet up, the chemical reaction takes place that hardens the fines instead of swelling them as clay will. Therefore, while cement typically attracts water, the intake of the water only produces a structural problem if the cement content is too high causing the structure to crack as it hardens due to the shrinking that naturally occurs in concrete as it cures.

Moisture intake in granular base is an issue since clay fines are typically used as a soil stabilizer enhancing cohesion of the material. Clay will swell and decrease the structural capacity of the material when wet. Therefore water intake values in granular base that are similar to those of RAP will have different effects on the aggregates.

The recycled aggregates, with the exception of the well graded PCC, have moisture intake values similar to conventional granular base. The high fines content of the well graded PCC are thought to be the cause of the higher moisture intake values. However, as most of the fines would be cement fines, this may not be a problem for the granular material, depending on the proportion of the fines being cement instead of clay.

Table 4.7 Moisture Intake Results for Granular Materials

	Water Intake
Granular Base	6.1%
19mm RAP (GW)	6.3%
25mm RAP (OGBC)	6.2%
CIP RAP	7.8%
19mm PCC (GW)	11.3%
19mm PCC (OGBC)	5.2%

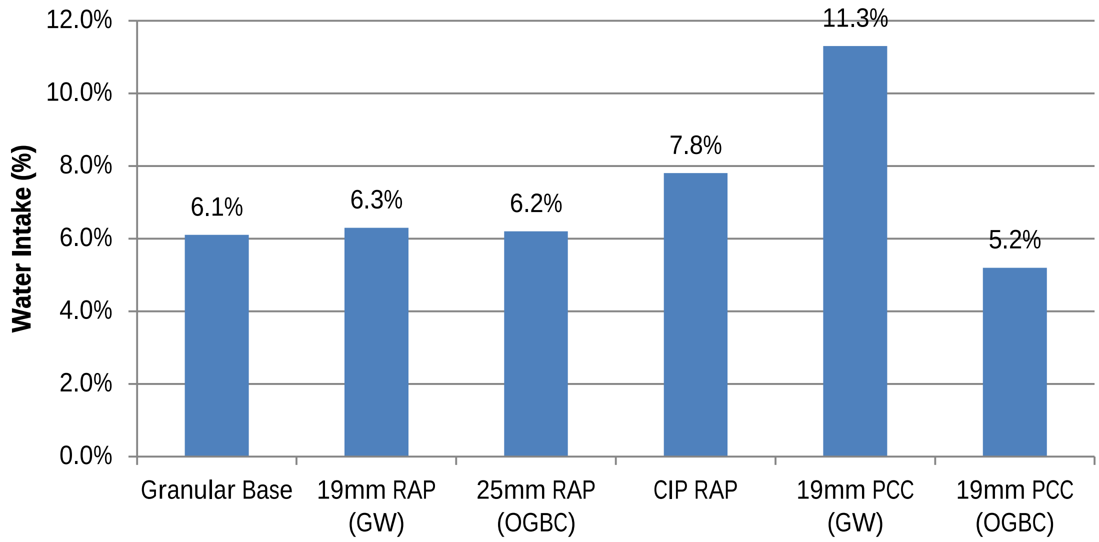


Figure 4.8 Moisture Intake Test Results for Granular Materials

4.6.2 Moisture Susceptibility

In addition to the moisture intake test as shown above, the granular materials were tested to determine how sensitive the materials are to moisture by using a dielectric probe. As shown in Table 4.8 and illustrated in Figure 4.9, when compared against each other, the open graded materials had far less electrical conductivity than the well graded aggregates, both recycled and pit derived. As the open graded aggregates material had significantly less fines than the well graded, this was expected.

Table 4.8 Electric Conductivity Results for Granular Materials

	Conductivity (µS/cm)
Granular Base	100
19mm RAP (GW)	90
25mm RAP (OGBC)	32
CIP RAP	198
19mm PCC (GW)	267
19mm PCC (OGBC)	8

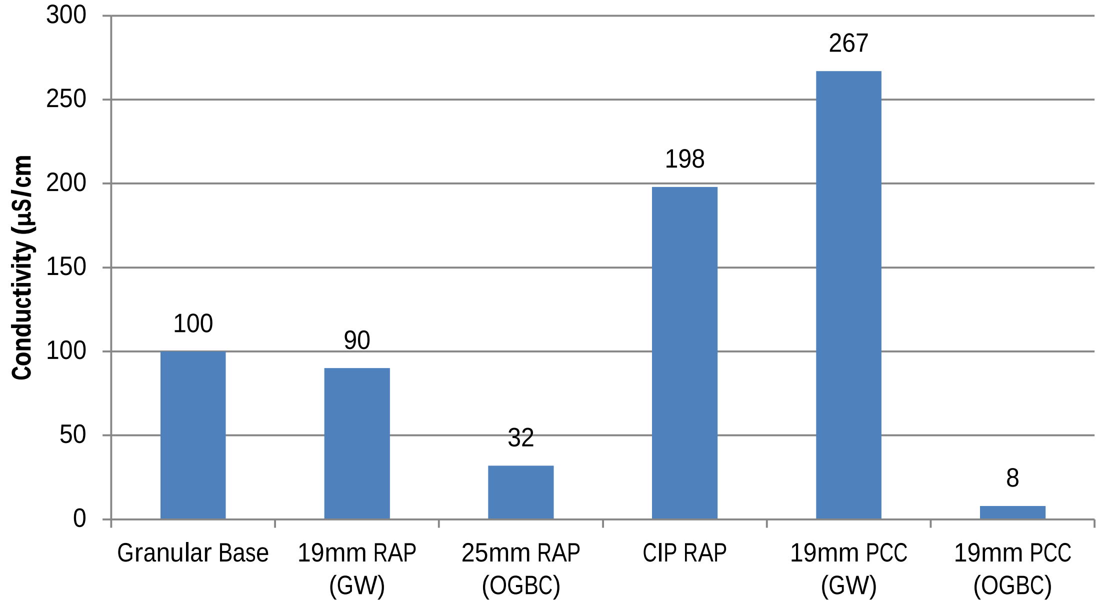


Figure 4.9 Electric Conductivity of Tested Aggregates

4.7 Chapter Summary

The recycled asphalt aggregates have a dynamic modulus of 200 percent higher than that of high fracture granular aggregates while the recycled portland cement concrete aggregates had a dynamic modulus of 80 to 100 percent greater than conventional high fracture granular aggregates. These higher dynamic moduli indicate that the recycled aggregates are more resistant to strain and therefore will either produce tougher structures at equal thicknesses or allow the reduction in granular thicknesses for equal structural capacity.

The recycled asphalt aggregates show increasing dynamic modulus as the traffic speed increases due to the viscous nature of asphalt. This same dynamic response to loading was not present in the PCC and conventional granular base samples.

The results from the Poisson’s Ratio indicate that recycled aggregates have improved structural mechanical characteristics when compared to the conventional high fracture granular base. The recycled portland cement concrete had a Poisson’s Ratio of 0.31 to 0.40 while the Poisson’s Ratio for recycled asphalt was 0.31 to 0.39. The high fracture granular base had the highest Poisson's ratio of 0.40 to 0.43 indicating more deformation under the high stress state than the recycled aggregates.

The recycled aggregates had an increased shear modulus when compared to a high fracture conventional base. The resistance to shear, especially with the 25mm RAP (OGBC) was 584% greater than high fracture base at high frequency and 266% greater than high fracture base at low frequency.

Creating an OGBC with the recycled asphalt and recycled portland cement concrete aggregates produced a granular material that had a significant reduction in moisture intake as well as lowering the electrical conductivity of the granular material. These attributes show a significant reduction in moisture sensitivity with the OGBC gradation when comparing to the standard base course gradation in the recycled aggregates.

CHAPTER 5 ROADWAY STRUCTURAL DESIGN

Structural design in the City of Saskatoon is currently performed based on historical highway roadway design (City of Saskatoon 2009, Thomas 2009). This chapter provides an outline as to how the current roadway designs are performed in the City of Saskatoon as well as providing an alternative design method based on mechanistic material characteristics.

5.1 Conventional City of Saskatoon Design

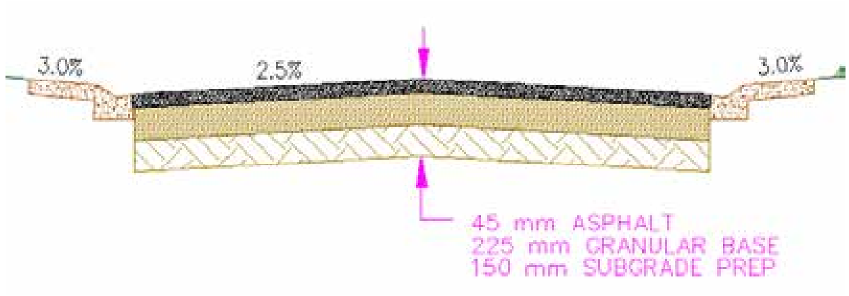
The City of Saskatoon roadway network was segmented into five classifications of roadways, depending on traffic loading, traffic volume and traffic type. The local classification was for residential streets and areas experiencing less than 1,500 ADT. Transit activity is prohibited on this road class for normal operations. The next larger designation is for Collector roadways where the roadway traffic volume is 1,000 to 12,000 ADT. The Industrial roadways designation is limited to industrial areas of the City and is specialized due to the higher volume of heavy truck traffic. Collector roadways feed into the City of Saskatoon Arterial roadway network. This network is designed for an ADT of between 5,000 and 30,000 ADT. The last road classification is for Freeway roadways designed for greater than 30,000 ADT.

As outlined in the neighbourhood planning guide, the City of Saskatoon will increase the thickness of the granular structure by 150mm in place of 150mm of subgrade preparation if the subgrade has a CBR of less than 5.0. This procedure is done across all road classes except expressways. These granular structures will vary from as low as 225mm for local roadways up to 450mm for the standard arterial structure. Each structure in a low subgrade CBR situation would have an additional 150mm of granular structure. The cross sections of the design local and arterial structures as outlined in Table 5.1 and are illustrated in Figure 5.1.

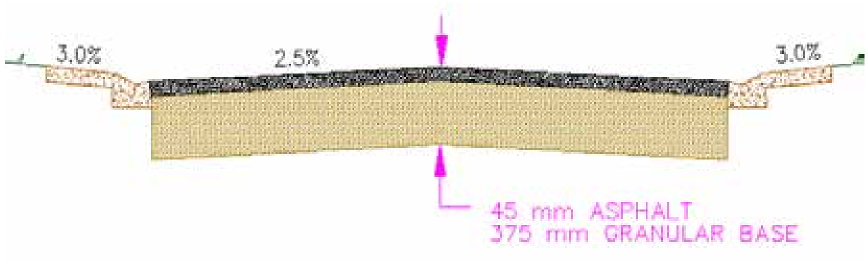
Table 5.1 Roadway Thickness Structures for City of Saskatoon

	Layer Thickness (mm)		
	Local	**Collector**	**Arterial**
Subgrade CBR >5			
HMA	45	80	100
Base	225	150	150
Sub-base	0	225	300
Subgrade Prep	150	300	300
Subgrade CBR<5			
HMA	45	80	100
Base	375	150	300
Sub-base	0	375	300
Subgrade Prep	0	0	0
Geotextile	no	no	yes

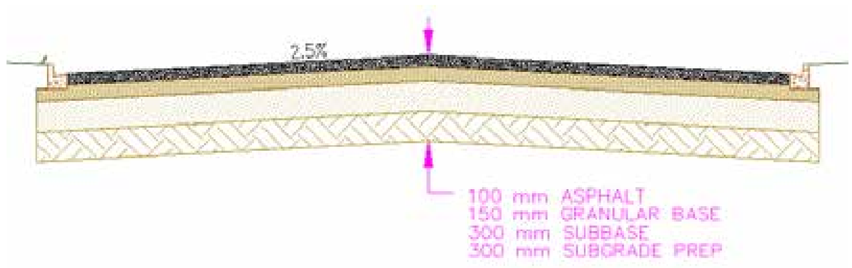
This methodology goes contrary to the Saskatchewan Highways methodology of preparing the subgrade to 100% of optimum standard proctor density up to a depth of 600mm. Highways design does this to have a proper working platform for the roadway construction and to keep the subgrade from failing during construction. The City of Saskatoon typically will build roadways after the underground utilities are constructed. The subgrade is compacted to 98% of optimum density during the backfill processes and is considered adequate for constructing the roadway.



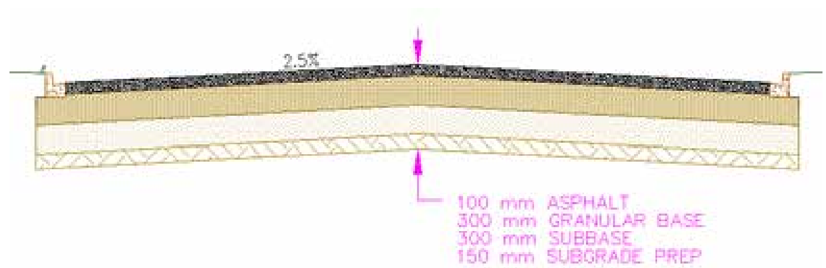
a) Typical COS Local Structure with Dry Subgrade



b) Typical COS Local Structure in Wet Subgrade



c) Typical COS Arterial Structure with Dry Subgrade



d) Typical COS Arterial Structure with Wet Subgrade

Figure 5.1 Typical COS Structures

5.2 Mechanistic Roadway Analysis

In order to perform mechanistic roadway design, a few alternatives were investigated. The software chosen for this work was the PSIPave3D™ system which incorporates the mechanistic material properties into a finite element model (FEM) and provides output in the strains experienced due to the programmed loading. Each of the conventional cross sections outlined in section 5.1 were modeled as well as other unconventional test sections using recycled material for the aggregates and including drainage layers to address the subsurface moisture issues. The FEM had an input screen as shown in Figure 5.2 where the various conventional aggregate properties can be entered and the cross-sections of the roadway can be defined.

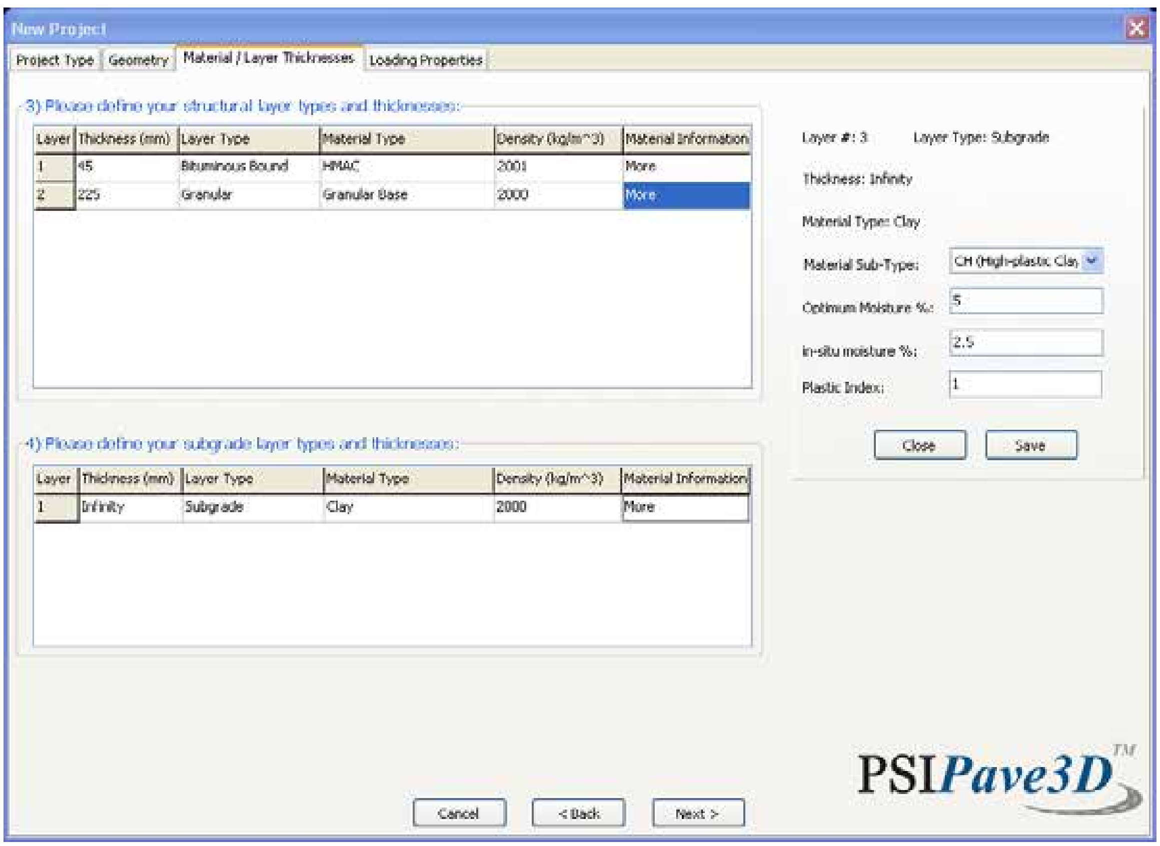


Figure 5.2 PSIPave3D™ Model Input Screen

The FEM then uses the conventional geotechnical and physical material properties commonly employed in road engineering and chooses the appropriate mechanistic characteristics to build the model. Using a defined loading scheme the strains on the structure can be modeled and output either as a graphic, shown in Figure 5.3, or as a data file. Complete data sets from the modeling are included in Appendix F.

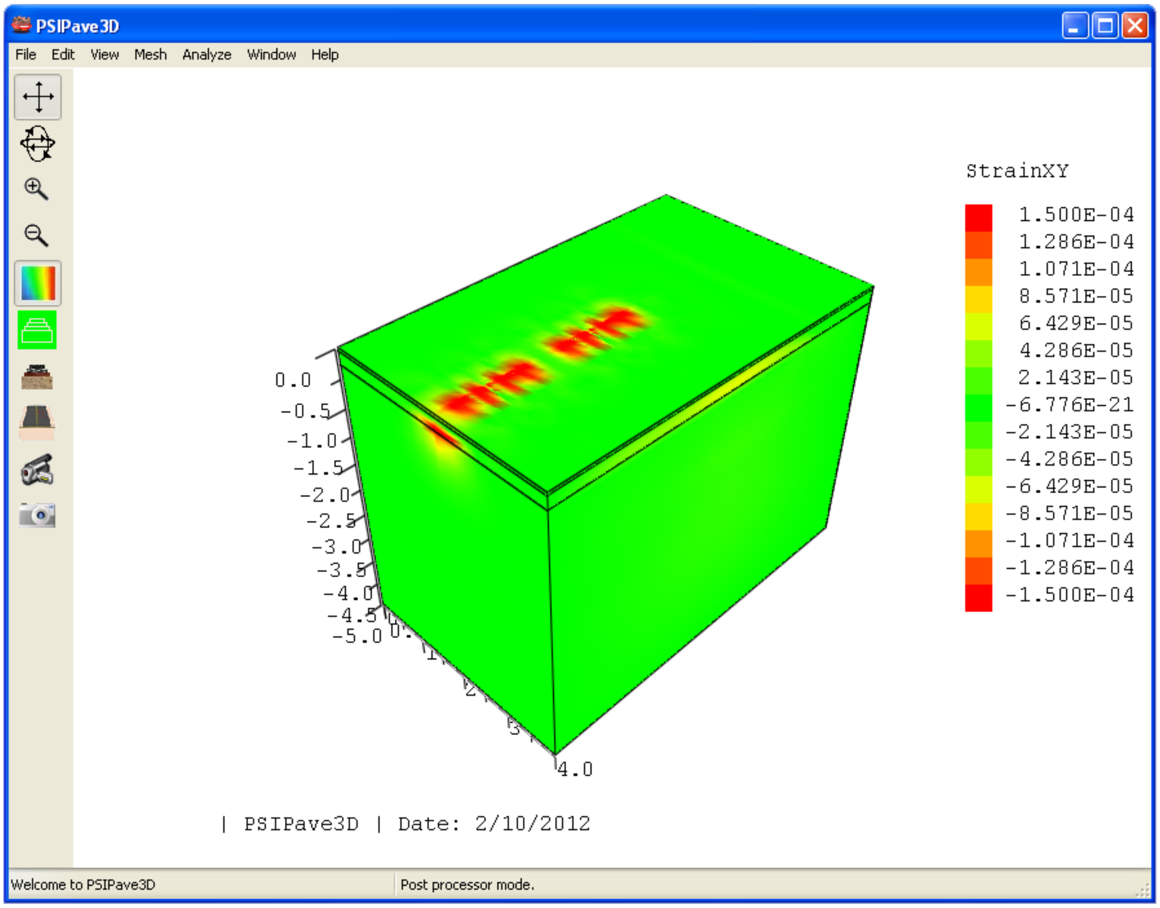
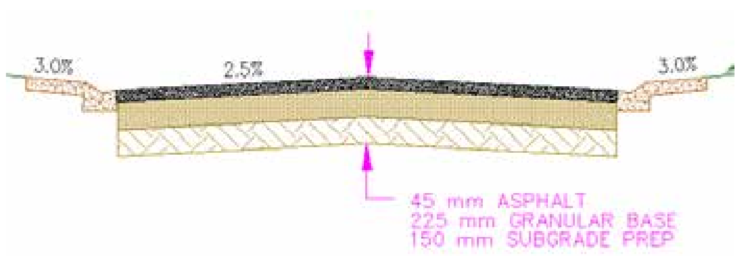


Figure 5.3 FEM Model Strain Output Graphic

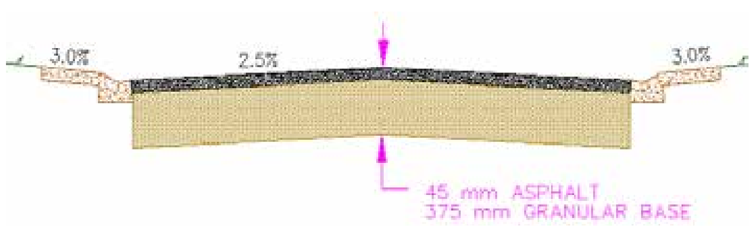
5.2.1 Local Road Structural Mechanistic Modelling

Local structures are designed to have very thin granular structures on a firm base, which requires strains in the subgrade to be below a maximum level. Given this thin structure, when critical loads are present, the subgrade has to take a significant share of the structural support. Typical conventional roadway design shows that this thickness will support the number of traffic loads that the roadway experiences. However, designing the roadways this thin causes the roadways to be highly susceptible to critical loadings and at risk during the freeze thaw cycles. Modelling the roadway structures based on their mechanistic responses to stresses will show the structures that will be at a high risk of failure due to critical loadings from various large vehicles that travel these roadways periodically. In order to attempt to compare similar structures, the local structures modeled include the same cross sections as outlined in section 5.1 and then substituting RAP for the granular base and adding a drainage layer with similar thicknesses. Due to constructability issues, the drainage layer has a minimum thickness of 225mm.

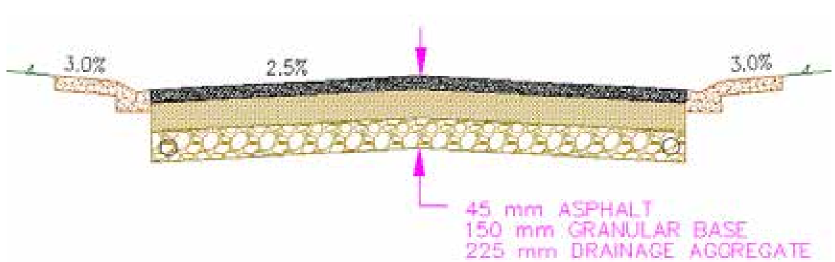
Table 5.2 and Figure 5.4 outline the local structures evaluated through the road structural modelling software. Figure 5.4 illustrates the cross section of the structures modeled on the two subgrades, highly plastic clay (CH) and silty clayey sand (SM-SC), and two moisture contents. Table 5.2 outlines which of the cross sections are modeled with each subgrade type and moisture content.



a) Baseline - Standard Local Structure



b) Thickened - Local Structure with Additional 150mm Granular



c) Drainage Structure - Local Structure with Drainage Layer

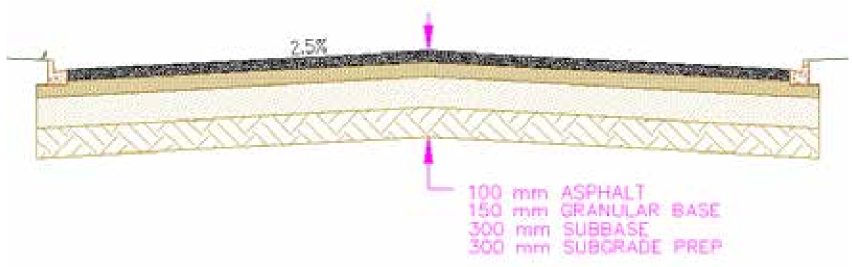
Figure 5.4 Modeled Local Structure Cross Sections

Table 5.2 Modeled Local Structure Cross Sections

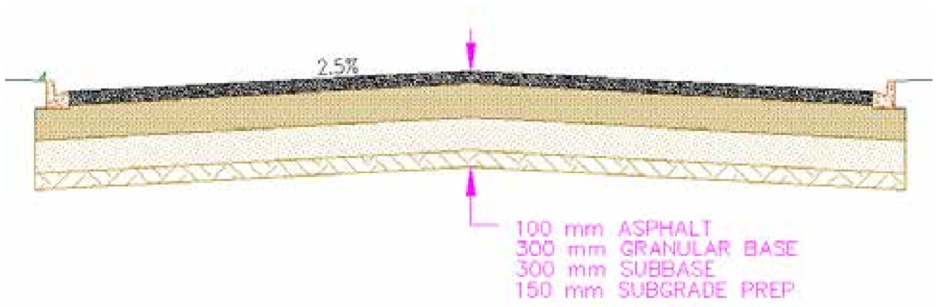
Modeled Structure	Subgrade Type	Subgrade Moisture	Granular Layer(s)	Asphalt
Subgrade and Moisture Content Variation				
SM-SC/-3%/225GB/45AC	**SM-SC**	Opt - 3% Dry	225mm GB	45mm AC
CH/-3%/225GB/45AC	**CH**	Opt - 3% Dry	225mm GB	45mm AC
SM-SC/+3%/225GB/45AC	**SM-SC**	Opt + 3% Wet	225mm GB	45mm AC
CH/+3%/225GB/45AC	**CH**	Opt + 3% Wet	225mm GB	45mm AC
Thickened Granular Layer				
CH/+3%/375GB/45AC	CH	Opt + 3% Wet	**375mm GB**	45mm AC
Granular Material Variation				
SM-SC/-3%/225RAP/45AC	SM-SC	Opt - 3% Dry	**225mm RAP**	45mm AC
CH/+3%/225RAP/45AC	CH	Opt + 3% Wet	**225mm RAP**	45mm AC
SM-SC/-3%/225GB(HF)/45AC	SM-SC	Opt - 3% Dry	**225mm GB (HF)**	45mm AC
Drainage Layer				
CH/+3%/225CR/150GB/45AC	CH	Opt + 3% Wet	**150mm GB, 225mm Crushed PCC**	45mm AC

5.2.2 Arterial Structural Modeling

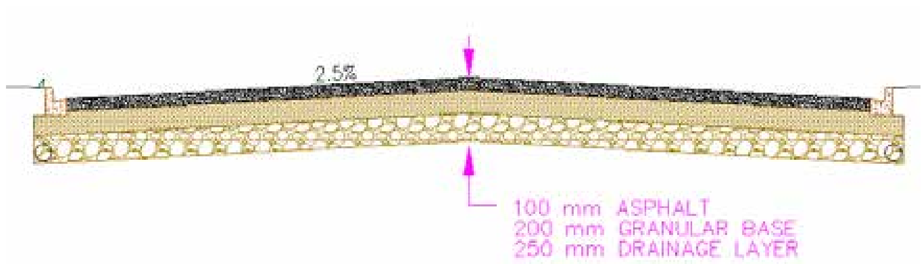
The City of Saskatoon arterial structures have a granular thickness of at least 450mm and are therefore less dependent on the subgrade for structural support. The various structures, both conventional and proposed, were modeled to determine the effect of the various granular materials and the subgrade properties. Table 5.3 and Figure 5.5 show the structures modeled in this section to compare their strain responses. Figure 5.5 illustrates the cross section of the structures modeled on the two subgrades, highly plastic clay (CH) and silty clayey sand (SM-SC), and two moisture contents. Table 5.3 outlines which of the cross sections are modeled with each subgrade type and moisture content.



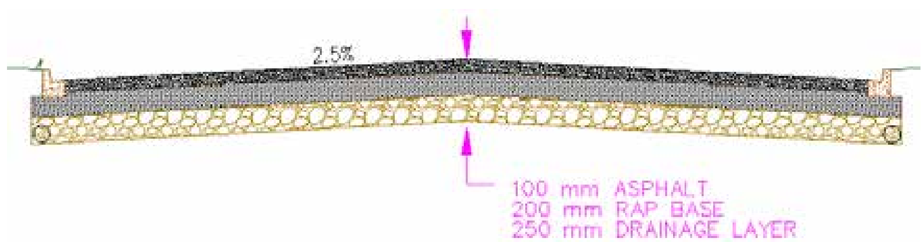
a) Baseline - Standard Arterial Structure



b) Thickened - Arterial Structure with Additional 150mm Granular



c) Drainage Structure - Arterial Structure with Granular Base and Drainage Layer



d) Drainage Structure (RAP) - Arterial Structure with RAP Base and Drainage Layer

Figure 5.5 Modeled Arterial Structure Cross Sections

Table 5.3 Modeled Arterial Structure Cross Sections

Modeled Structure	Subgrade Type	Subgrade Moisture	Granular Layers	Asphalt
Subgrade and Moisture Content Variation				
SM-SC/-3%/ 300SB/150GB/ 100AC	**SM-SC**	**Opt - 3% Dry**	150mm GB 300mm SB	100mm AC
CH/-3%/ 300SB/150GB/ 100AC	**CH**	**Opt - 3% Dry**	150mm GB 300mm SB	100mm AC
SM-SC/+3%/ 300SB/150GB/ 100AC	**SM-SC**	**Opt + 3% Wet**	150mm GB 300mm SB	100mm AC
CH/+3%/ 300SB/150GB/ 100AC	**CH**	**Opt + 3% Wet**	150mm GB 300mm SB	100mm AC
Thickened Granular Layer				
CH/+3%/ 450SB/150GB/ 100AC	CH	Opt + 3% Wet	**300mm GB** 300mm SB	100mm AC
Granular Material Variation				
SM-SC/-3%/ 300SB/150RAP/ 100AC	SM-SC	Opt - 3% Dry	**150mm RAP** 300mm SB	100mm AC
CH/+3%/ 300SB/150RAP/ 100AC	CH	Opt + 3% Wet	**150mm RAP** 300mm SB	100mm AC
SM-SC/-3%/ 300SB/150GB(HF)/ 100AC	SM-SC	Opt - 3% Dry	**150mm GB (high fines)** 300mm SB	100mm AC
Drainage Layer				
CH/+3%/ 250CR/200GB/ 100AC	CH	Opt + 3% Wet	**200mm GB 250mm Crushed PCC**	100mm AC
CH/+3%/ 250CR/200RAP/ 100AC	CH	Opt + 3% Wet	**200mm RAP 250mm Crushed PCC**	100mm AC

5.2.3 Subgrade Sensitivity

A primary factor that affects how the roadway structure performs is the *in situ* subgrade. As such, the CBR charts used by SMHI and the City of Saskatoon are created for varying types of subgrades (Saskatchewan Ministry of Highways and Infrastructure, 2009). In order to determine the effect of the subgrade type, two types of subgrade were modeled for structural response in their wet and dry states.

5.2.3.1 Local Roadway Structure

With the City of Saskatoon local structures being relatively thin the expected response is higher strains when the subgrade weakens. Modelling the structures in wet and dry conditions, as shown in Table 5.4 and illustrated in Figure 5.6, show the classic strain responses used to evaluate the roadways in the Shell curves. Highly plastic clay (CH) subgrades have close to double the compressive strain on the top of the subgrade when compared to the silty clayey sand (SM-SC) subgrade when dry. Having the subgrade wet instead of dry increased the strain on the top of the subgrade 804 percent for the SM-SC and 500 percent for the CH subgrade. The tensile strain on the bottom of the AC layer is similar for both subgrade types with the SM-SC having a 3.7 percent higher tensile strain in the bottom of the AC layer when the subgrades are dry and the CH having a 11.2 percent higher tensile strain when the subgrades are wet. When the subgrade gets wet, the tensile strain on the bottom of the AC increased 481% on CH subgrade and 417 percent on SM-SC subgrade.

Table 5.4 Local Structural Strain Comparisons with Different Subgrades

Modeled Structure	Max Tensile Strain at Bottom of AC	Compressive Strain on Top of Subgrade
Dry Subgrade		
SM-SC/-3%/225GB/45AC	310	493
CH/-3%/225GB/45AC	299	924
Wet Subgrade		
SM-SC/+3%/225GB/45AC	1295	3966
CH/+3%/225GB/45AC	1440	4617

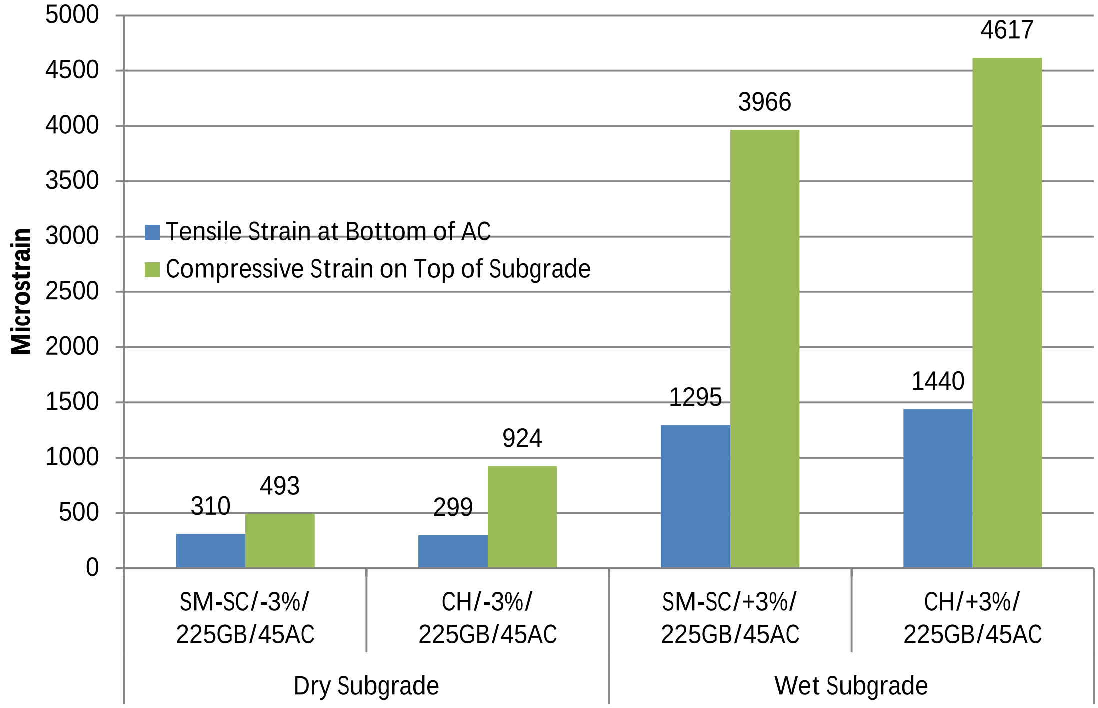


Figure 5.6 Local Structural Strain Comparisons with Different Subgrades

The structure was also evaluated to determine the shear stresses experienced in the various layers. The standard methods to evaluate the roadway assume that the tensile strain at the bottom of the asphalt is greater than the shear that the asphalt will experience. As well, to keep roadway design calculable, the shears within the granular structure and subgrade are assumed to be less than the horizontal and vertical strains. Computing power is now available to model the 3D strains within the roadway and calculate the strains. The side by side comparison of the local structure on a CH subgrade in dry and wet conditions indicated in Figure 5.7 shows the impact of moisture conditions on the shear strain in a roadway structure. The high shear strains, shown in red, in the wet structure will bring the roadway to failure much sooner than its design life.

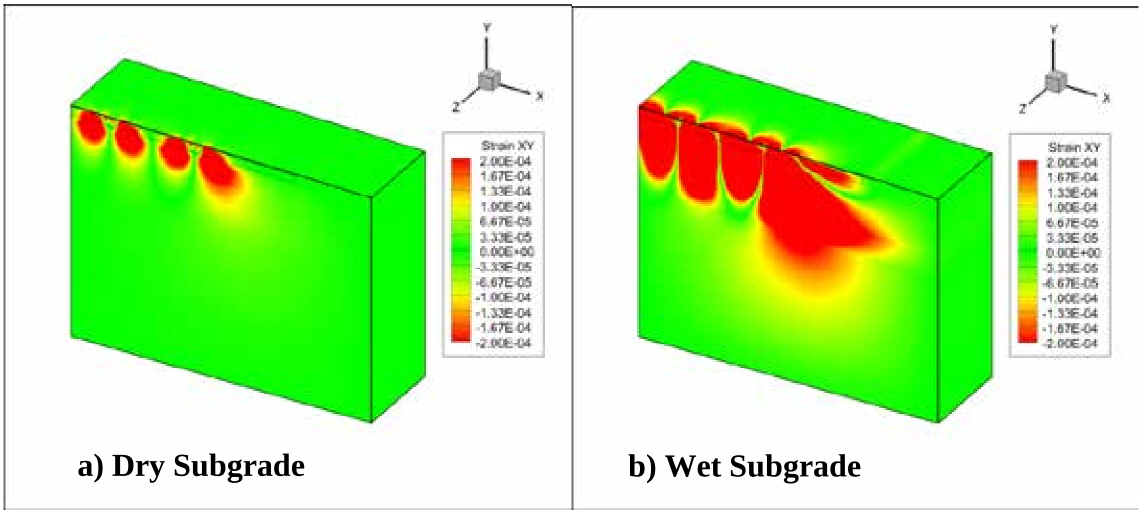


Figure 5.7 Dry versus Wet Local Roadway Shear Strain Comparison

Table 5.5 and Figure 5.8 show the maximum shear strains experienced in each of the roadway layers. As shown, the shear strains are 17 percent to 113 percent greater in the asphalt layer than the tensile strain illustrated in Figure 5.6. The CH structure had higher shear strains than the SM-SC structure of 3.6 percent, 9.1 percent and 66.7 percent in the AC, granular and subgrade, respectively, when the subgrade was dry of optimum. The shear strains increased between 253 percent and 785 percent when the subgrades were modeled as wet of optimum. The CH structure had higher shear strains than the SM-SC structure of 9.1 percent, 17.1 percent and 36.5 percent in the AC, granular and subgrade, respectively, when the subgrade was wet of optimum.

Table 5.5 Local Roadway Shear Strain Comparisons across Subgrade Type and Moisture Content

	Maximum Shear Strain by Layer		
Modeled Structure	**AC**	**Granular Layer**	**Subgrade**
Dry Subgrade			
SM-SC/-3%/225GB/45AC	615	1108	445
CH/-3%/225GB/45AC	637	1209	742
Wet Subgrade			
SM-SC/+3%/225GB/45AC	1556	4309	3493
CH/+3%/225GB/45AC	1697	5044	4767

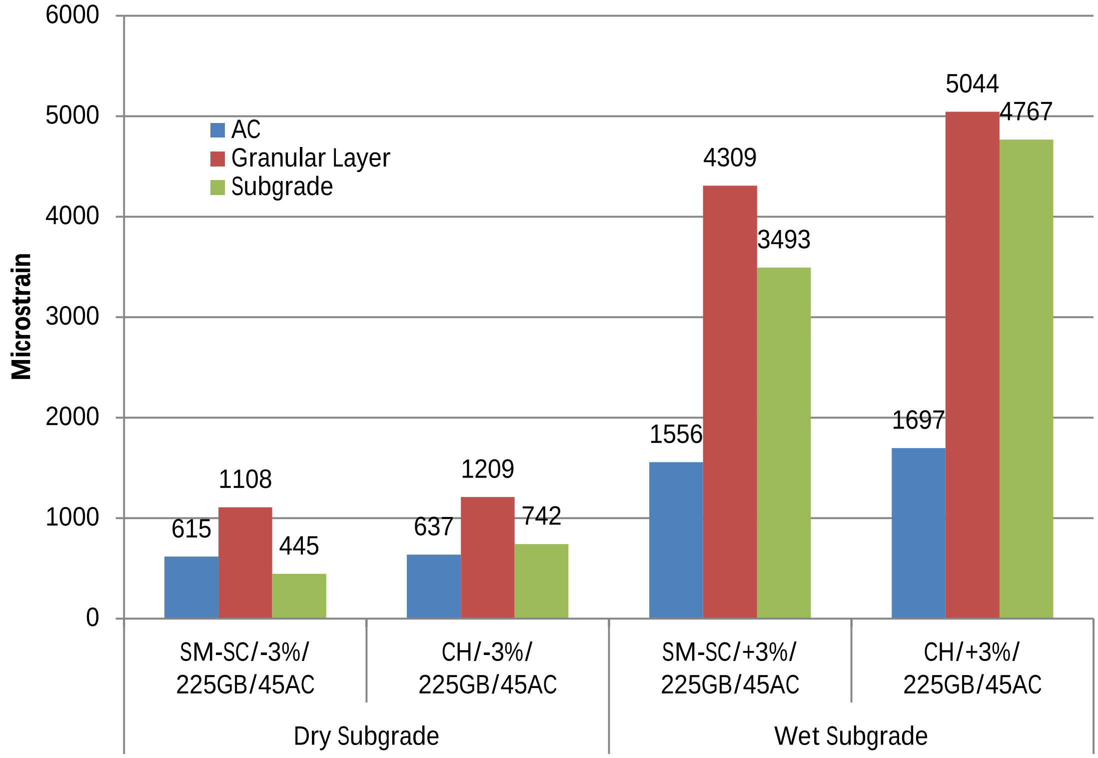


Figure 5.8 Local Roadway Shear Strain Comparisons across Subgrade Type and Moisture Content

5.2.3.2 Arterial Road Structures

While the arterial structures are thicker than the local structures, they are still quite dependant on subgrade type and moisture conditions as shown in Table 5.6 and illustrated in Figure 5.9. Highly plastic clay (CH) subgrades have close to double the compressive strain on the top of the subgrade when compared to the silty clayey sand (SM-SC) subgrade when dry. Having the subgrade wet instead of dry increased the strain on the top of the subgrade 718 percent for the SM-SC and 425 percent for the CH subgrade. The tensile strain on the bottom of the AC layer is similar for both subgrade types with the SM-SC having a 0.4 percent higher tensile strain in the bottom of the AC layer when the subgrades are dry and the CH having a 6.1 percent higher tensile strain when the subgrades are wet.

Table 5.6 **Arterial Structural Strain Comparisons with Different Subgrades**

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Dry Subgrade		
SM-SC/-3%/300SB/150GB/100AC	251	205
CH/-3%/300SB/150GB/100AC	250	386
Wet Subgrade		
SM-SC/+3%/300SB/150GB/100AC	707	1471
CH/+3%/300SB/150GB/100AC	750	1641

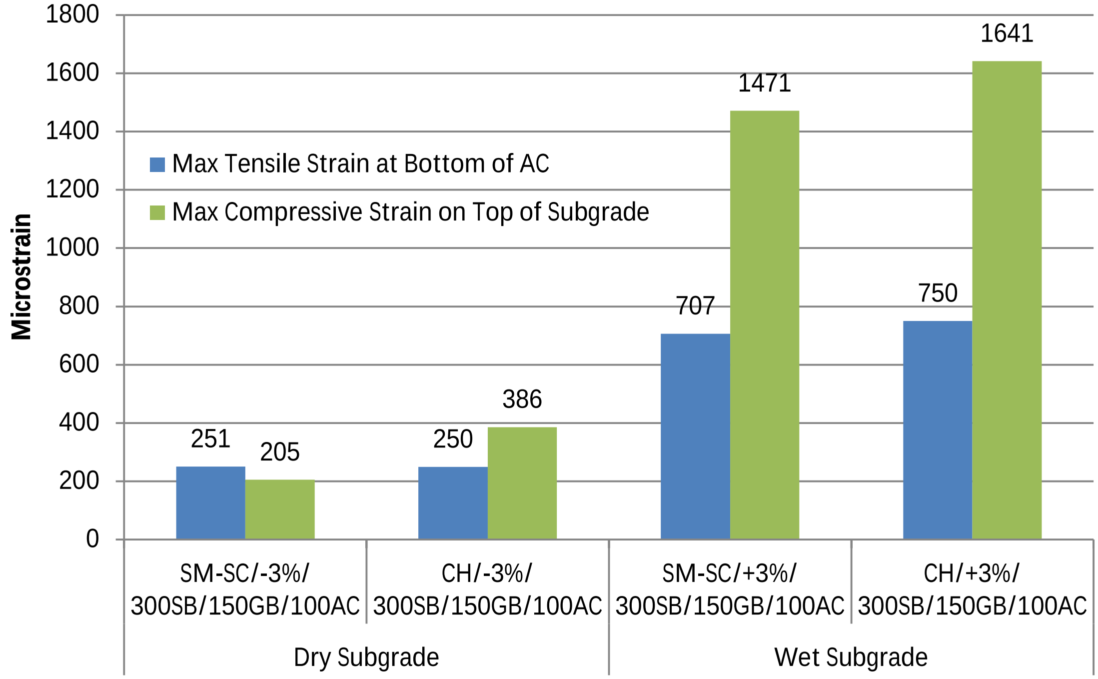


Figure 5.9 Arterial Structural Strain Comparisons with Different Subgrades

When evaluating the shear across the various layers with the FEM, the shear strains are higher than the tensile and compressive strains shown in Figure 5.9 when the subgrades are dry. As indicated in Table 5.7 and illustrated in Figure 5.10, the shears also increase in all layers when the subgrades are wet. The CH structure had higher shear strains than the SM-SC structure of 2.0 percent, 4.6 percent, 9.6 percent and 73.5 percent in the AC, granular, sub-base and subgrade, respectively, when the subgrade was dry of optimum. The shear strains increased between 190 percent and 719 percent when the subgrades were modeled as wet of optimum. The

CH structure had higher shear strains than the SM-SC structure of 4.3 percent, 22.6 percent, 20.2 percent and 33.8 percent in the AC, granular, sub-base and subgrade, respectively, when the subgrade was wet of optimum.

Table 5.7 Arterial Shear Strain Comparisons with Different Subgrades

Modeled Structure	Maximum Shear Strain by Layer			
	AC	Granular Layer	Sub-base/ Drainage	Subgrade
Dry Subgrade				
SM-SC/-3%/300SB/150GB/100AC	400	585	394	162
CH/-3%/300SB/150GB/100AC	408	612	432	281
Wet Subgrade				
SM-SC/+3%/300SB/150GB/100AC	760	1590	1630	1165
CH/+3%/300SB/150GB/100AC	793	1950	1960	1559

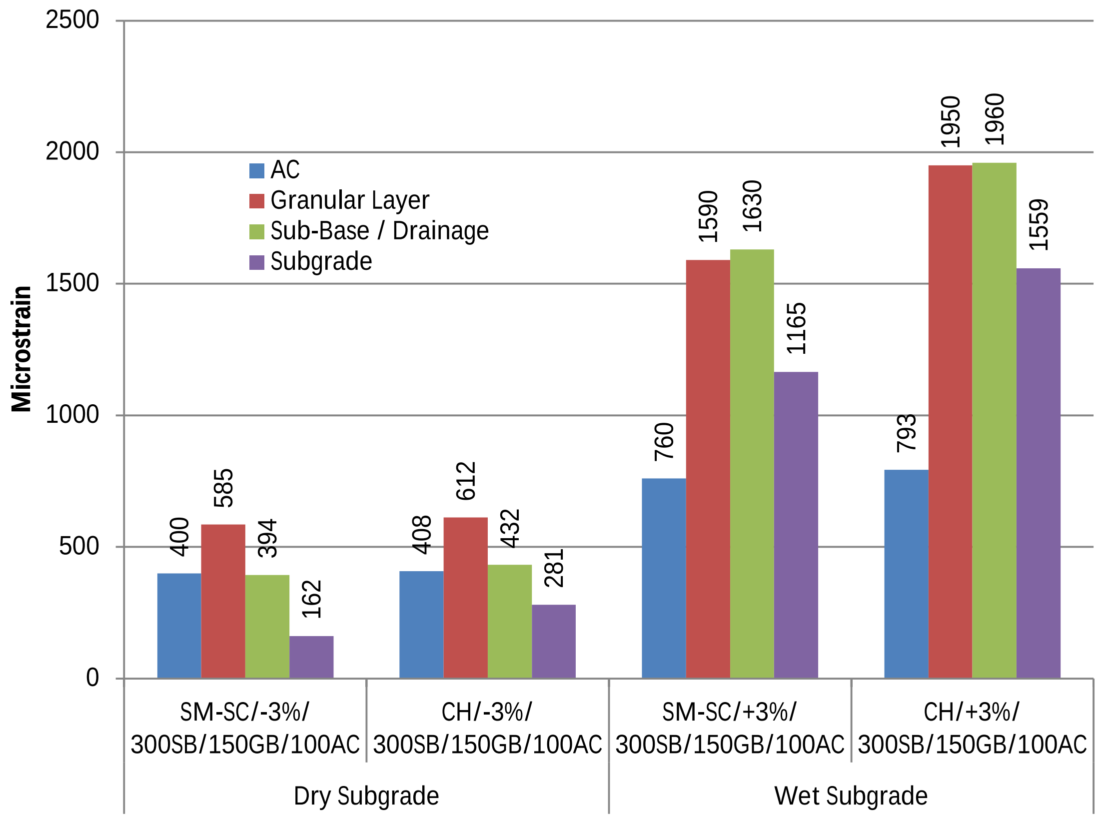


Figure 5.10 Arterial Shear Strain Comparisons with Different Subgrades

5.2.4 Granular Thickness Sensitivity

One of the current City of Saskatoon methods of designing roadways in wet areas is to excavate an additional 150mm of the subgrade and then replace with granular. The extra granular helps provide a working platform in which to pave. However, the long term strength of the structure is not known due to the impact of moisture on the subgrade and the granular layer. As the thickening only occurs when the subgrade is wet and has a soaked CBR of less than 5.0, the condition modeled was the wet highly plastic clay subgrade.

5.2.4.1 Local Roadway Structure

When the thickened structure is modeled in the typical local structure, as shown in Table 5.8 and illustrated in Figure 5.11, the FEM results indicate that increasing the granular layer by 150mm does reduce the strains in the subgrade by 21.3 percent and reduces the tensile strain in the bottom of the AC layer by 4.9 percent.

Table 5.8 Strain Comparisons with Different Granular Thicknesses

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Baseline CH/+3%/225GB/45AC	1440	4617
Thickened CH/+3%/375GB/45AC	1369	3634

When the shear strains are evaluated for the same scenarios, as shown in Table 5.9 and illustrated in Figure 5.12, a similar relationship can be seen. Shear strains are reduced in the asphalt layer and in the granular layer by 4.0 percent and 2.6 percent respectively, when the granular layer is thickened. There is a 30.5 percent reduction in the maximum shear experienced in the subgrade layer.

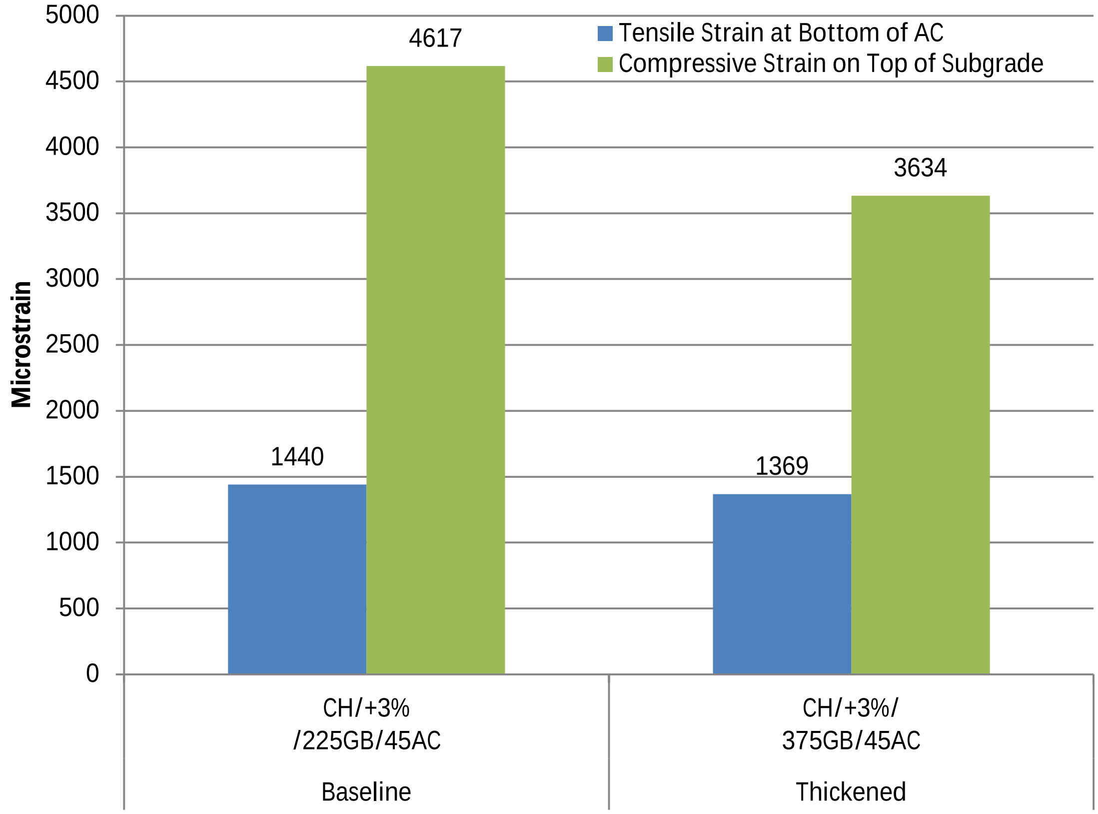


Figure 5.11 Local Structural Strain Comparisons with Different Granular Thicknesses

Table 5.9 Local Shear Strain Comparison across Thicknesses

Modeled Structure	**Maximum Shear by Layer**		
	AC	**Granular Layer**	**Subgrade**
Baseline CH/+3%/225GB/45AC	1697	5044	4767
Thickened CH/+3%/375GB/45AC	1629	4915	3314

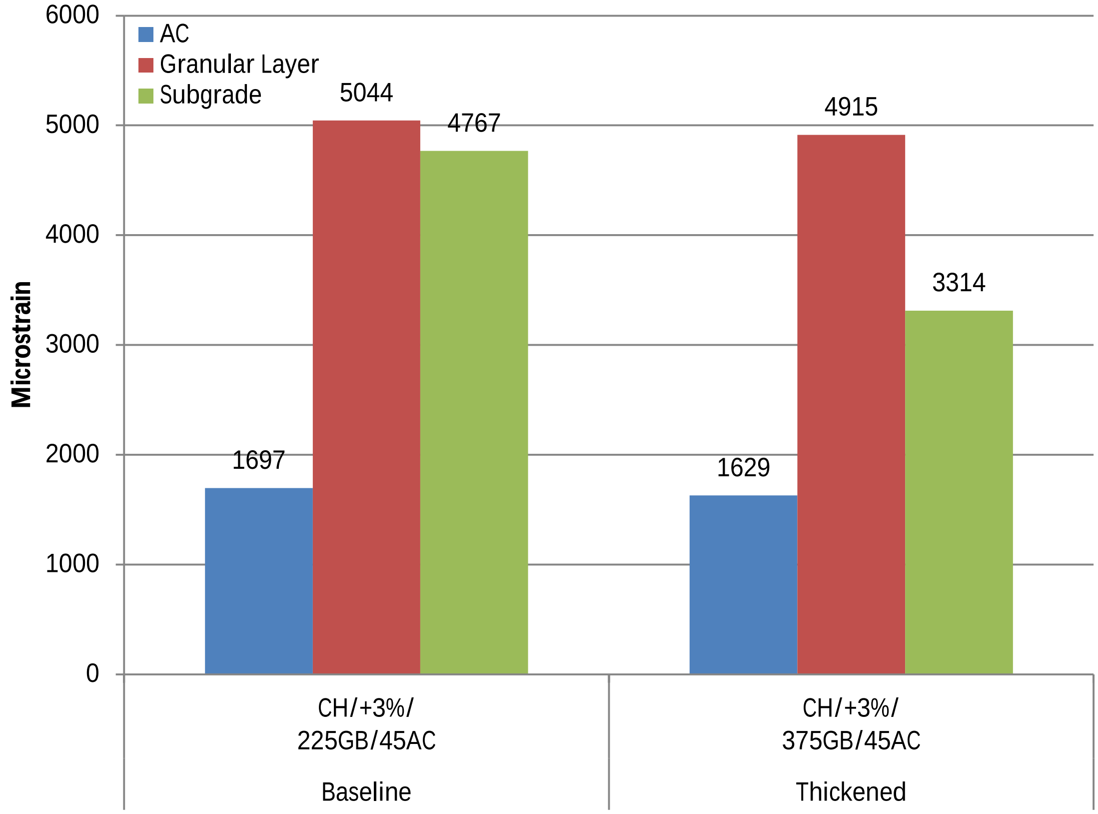


Figure 5.12 Local Shear Strain Comparison across Thicknesses

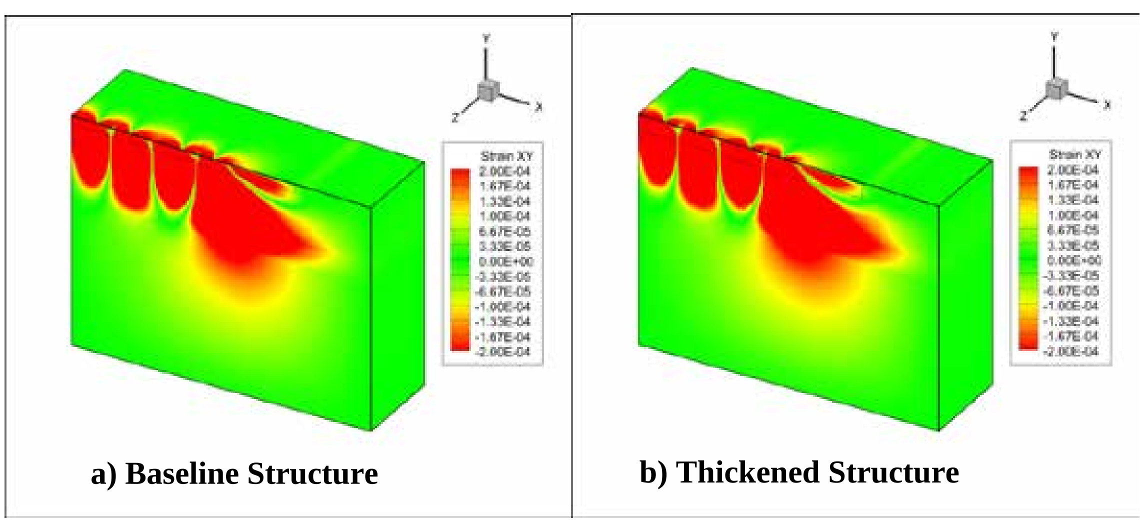


Figure 5.13 Baseline versus Thickened Local Roadway Shear Strain Comparison

The side by side comparison of the local structure, both standard thickness and with an additional 150mm of granular, on a CH subgrade in wet conditions indicated in Figure 5.13 shows the impact of thickening the road structure on the shear strain in a roadway structure. The thicker structure on the right indicates nearly identical shear characteristics to the standard structure.

5.2.4.2 Arterial Roadway Structure

The granular thickening approach is used for arterials as well in an attempt to protect the soft subgrade. As shown in Table 5.10 and illustrated in Figure 5.14 the thicker layer decreased the compressive strain on the top of the subgrade by 27.3 percent and reduced the tensile strain experienced at the bottom of the AC layer by 2.5 percent. Thickening the granular structure on arterial structures compared to local structures is more effective in reducing strains due to the subgrades having less impact on the structural integrity.

Table 5.10 Arterial Strain Comparisons with Different Granular Thicknesses

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Baseline CH/+3%/300SB/150GB/100AC	750	1641
Thickened CH/+3%/450SB/150GB/100AC	731	1194

The thickening of the granular layer does reduce the shear strains experienced in the lower granular layer as shown in Table 5.11 and illustrated in Figure 5.15. The shear strain in the subgrade increased slightly with the increase in granular thickness to become the dominant strain on the subgrade, negating any benefit shown in the conventional design criteria of Figure 5.16. Shear strains are reduced in the asphalt layer, granular layer and sub-base layer by 1.8 percent, 0.8 percent and 21.3 percent respectively, when the granular layer is thickened. However, the shear strain in the subgrade layer increases by 5.2 percent.

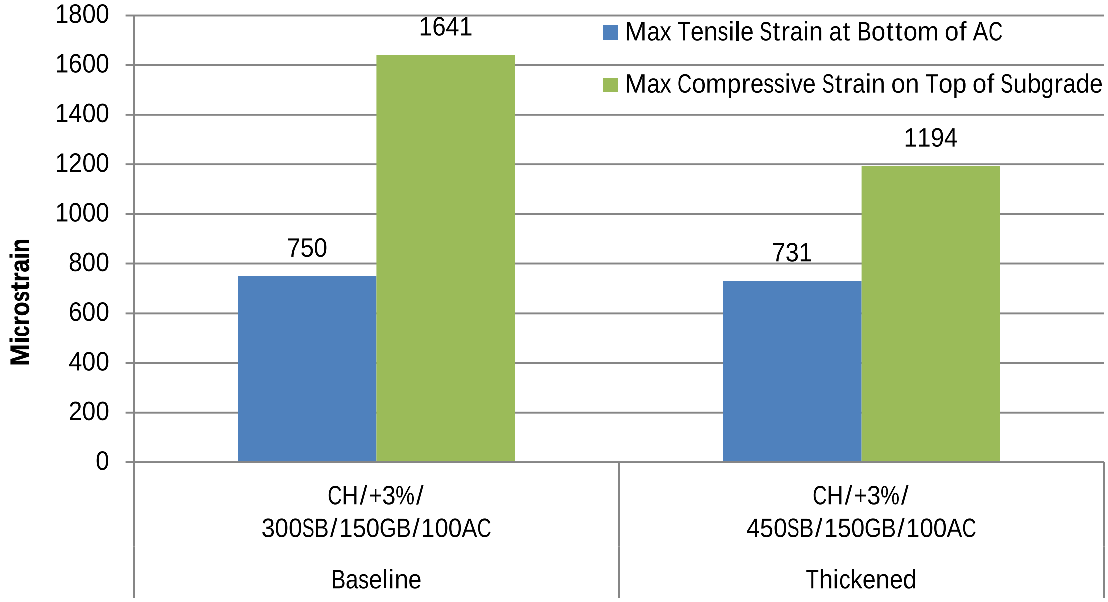


Figure 5.14 Arterial Structural Strain Comparisons with Different Granular Thicknesses

Table 5.11 Arterial Shear Strain Comparisons with Different Granular Thicknesses

Modeled Structure	Maximum Shear by Layer			
	AC	**Granular Layer**	**Sub-Base/ Drainage**	**Subgrade**
Baseline CH/+3%/300SB/150GB/100AC	793	1950	1960	1559
Thickened CH/+3%/450SB/150GB/100AC	779	1934	1542	1640

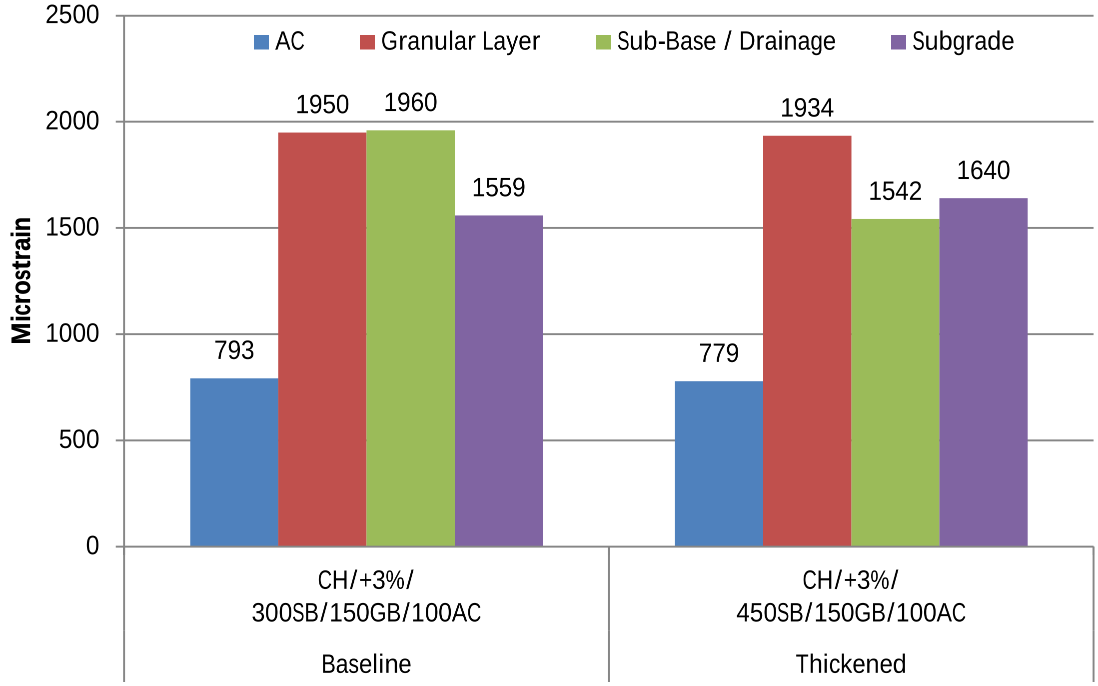


Figure 5.15 Arterial Shear Strain Comparisons with Different Granular Thicknesses

5.2.5 Base Granular Material Sensitivity

The City of Saskatoon allows between 6 and 11 percent of the granular base to be finer than 71 µm. As the amount of fines significantly affect the performance of the granular base (Guenther et al 2011), the sensitivity to base material type includes a high fines granular base. RAP material was included within the base granular material sensitivity evaluation as RAP was used in a number of test sections constructed by the City of Saskatoon.

5.2.5.1 Local Roadway Structure

The modeled response for this substitution as shown in Table 5.12 and illustrated in Figure 5.16 indicate that RAP performs better in dry and wet conditions than either standard granular base (GB) or high fines granular base (GB(HF)). The high fines granular base was produced by increasing the fine content to the maximum allowed by the City of Saskatoon specifications. In the dry condition the GB and GB(HF) had 276 percent and 308 percent higher tensile strain on the bottom of the AC, respectively, than the RAP structure. The compressive strain on the top of the subgrade for the GB and GB (HF) structures increased by 125 percent and 146 percent, respectively, when compared to the RAP structure. Comparing the strains in the wet subgrade between the RAP and GB structures indicates that the RAP reduces the tensile strain on the bottom of AC by 88.6 percent and the compressive strain on the top of the subgrade by 78.3 percent.

Table 5.12 Local Structural Strain Comparisons with Different Granular Materials across Dry and Wet Subgrades

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Dry Sand Subgrade		
SM-SC/-3%/225RAP/45AC	112	394
SM-SC/-3%/225GB(HF)/45AC	346	574
SM-SC/-3%/225GB/45AC	310	493
Wet Clay Subgrade		
CH/+3%/225RAP/45AC	164	1002
CH/+3%/225GB/45AC	1440	4617

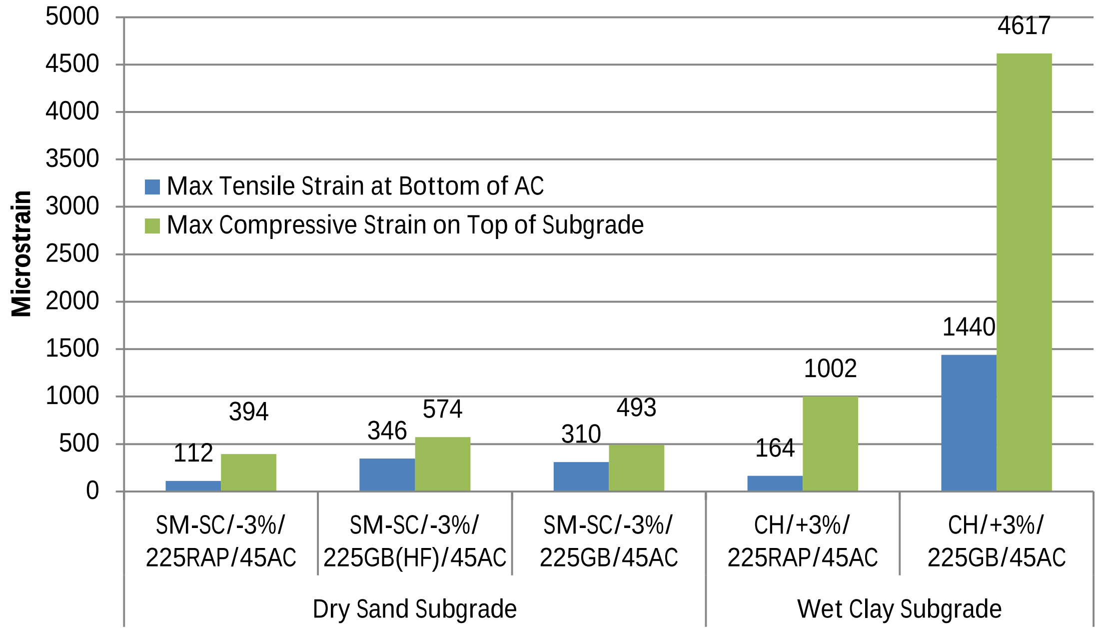


Figure 5.16 Local Structural Strain Comparisons with Different Granular Materials

The shear strains shown in Table 5.13 and illustrated in Figure 5.17 follow the same trend shown in Figure 5.16 with the model including RAP having the lowest shear strains. In all cases the maximum shear strain identified in the asphalt layer is greater than the tensile strain indicated in Figure 5.15. As well, the RAP versus granular base aggregate comparison in the dry sand subgrade shows the RAP reduces the shear strains experienced in the asphalt, granular and subgrade layers by 36.9 percent, 50.9 percent and 25.2 percent respectively. The high fines granular base (GB (HF)) increased the shear strain on the AC layer by 0.5 percent and the subgrade layer by 15.1 percent when compared to the granular base. When comparing the structures on wet clay subgrades the RAP reduces the shear strains experienced in the asphalt, granular and subgrade layers by 73.0 percent, 85.2 percent and 81.8 percent respectively.

To illustrate the impact of using RAP as the granular aggregate in a local structure, the two structures are shown side by side in Figure 5.18. The structure on the left is the standard base structure on a wet clay subgrade while the structure on the right is the RAP structure on a wet clay subgrade. It can be seen that the shear strains are significantly less in the RAP structure than the base granular structure.

Table 5.13 Local Shear Strain Comparisons with Different Granular Materials

Modeled Structure	Maximum Shear by Layer		
	AC	**Granular Layer**	**Subgrade**
Dry Sand Subgrade			
SM-SC/-3%/225RAP/45AC	388	544	333
SM-SC/-3%/225GB(HF)/45AC	618	1107	512
SM-SC/-3%/225GB/45AC	615	1108	445
Wet Clay Subgrade			
CH/+3%/225RAP/45AC	458	749	866
CH/+3%/225GB/45AC	1697	5044	4767

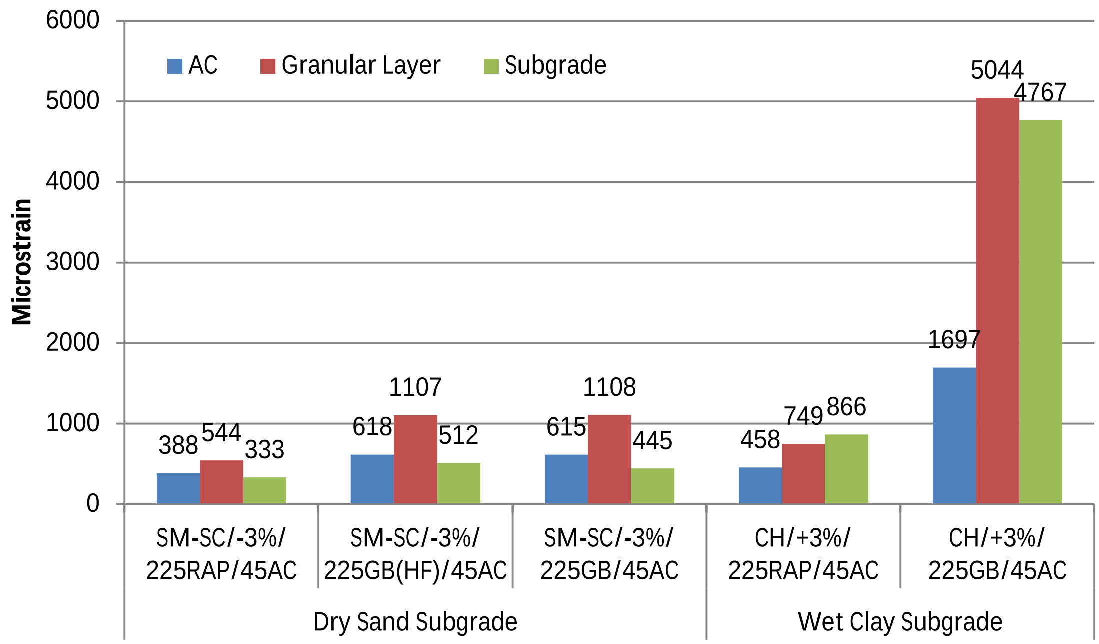


Figure 5.17 Local Shear Strain Comparisons with Different Granular Materials

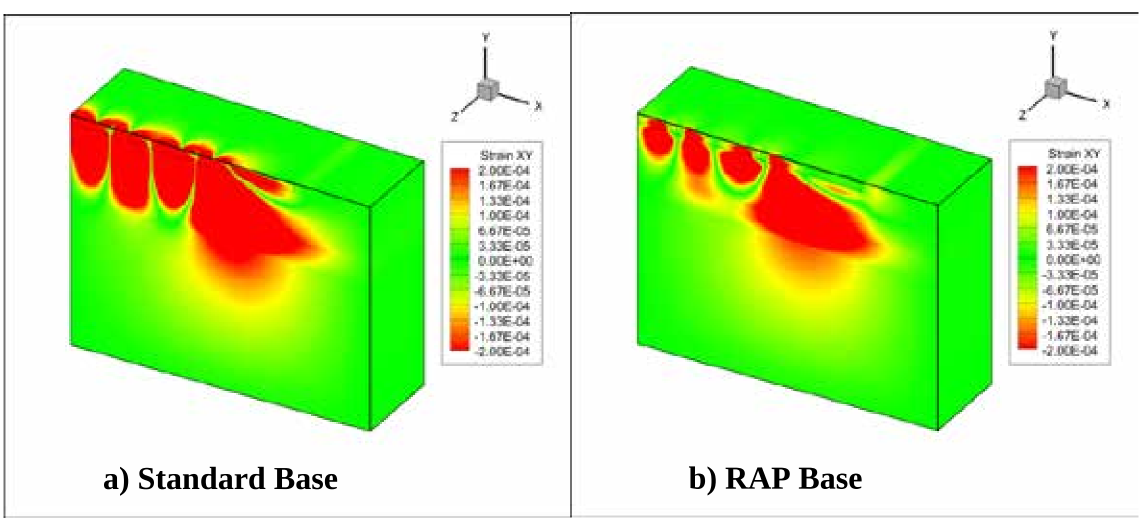


Figure 5.18 Standard versus RAP Local Roadway Shear Strain Comparison

5.2.5.2 Arterial Roadway Structure

In the arterial structures, the impact of using different granular materials in the top 150mm is equally significant. The structures were compared in dry and wet conditions with the top granular material as RAP, standard granular base aggregate (GB) and a high fines base aggregate (GB(HF)). When evaluating the RAP using its mechanistic material characteristics, the RAP performs better than standard base aggregates in both the dry and wet condition as shown in Table 5.14 and illustrated in Figure 5.19. In the dry condition the GB and GB(HF) had 85 percent and 101 percent, respectively, higher tensile strain on the bottom of the AC over the RAP structure. The compressive strain on the top of the subgrade for the GB and GB (HF) structures increased by 14.5 percent and 39.8 percent, respectively, over the RAP structure. Comparing the strains in the wet subgrade between the RAP and GB structures indicates that the RAP reduces the tensile strain on the bottom of AC by 78.8 percent and the compressive strain on the top of the subgrade by 52.8 percent.

Table 5.14 Arterial Structural Strain Comparisons with Different Granular Materials

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Dry Sand Subgrade		
SM-SC/-3%/300SB/150RAP/100AC	136	179
SM-SC/-3%/300SB/150GB(HF)/100AC	273	251
SM-SC/-3%/300SB/150GB/100AC	251	205
Wet Clay Subgrade		
CH/+3%/300SB/150RAP/100AC	159	775
CH/+3%/300SB/150GB/100AC	750	1641

A similar picture can be seen with the maximum shear strains in the various layers when evaluating across different granular materials as shown in Table 5.15 and illustrated in Figure 5.20. In all cases the maximum shear strain identified in the asphalt layer is greater than the tensile strain indicated in Figure 5.19. When evaluated on a dry sand subgrade the RAP structure reduced the shear strain in the AC, granular, sub-base and subgrade by 23.9 percent, 37.2 percent, 24.4 percent and 30 percent respectively in relation to the GB structure. The GB (HF) structure on the same dry sand subgrade had shear strains between 1.5 percent and 23.5 percent higher than the GB structure.

The RAP structure, when compared to the HF structure, on a wet clay subgrade reduced the shear strains in the AC, granular, sub-base and subgrade by 49.6 percent, 74.0 percent, 58.3 percent and 48.8 percent, respectively.

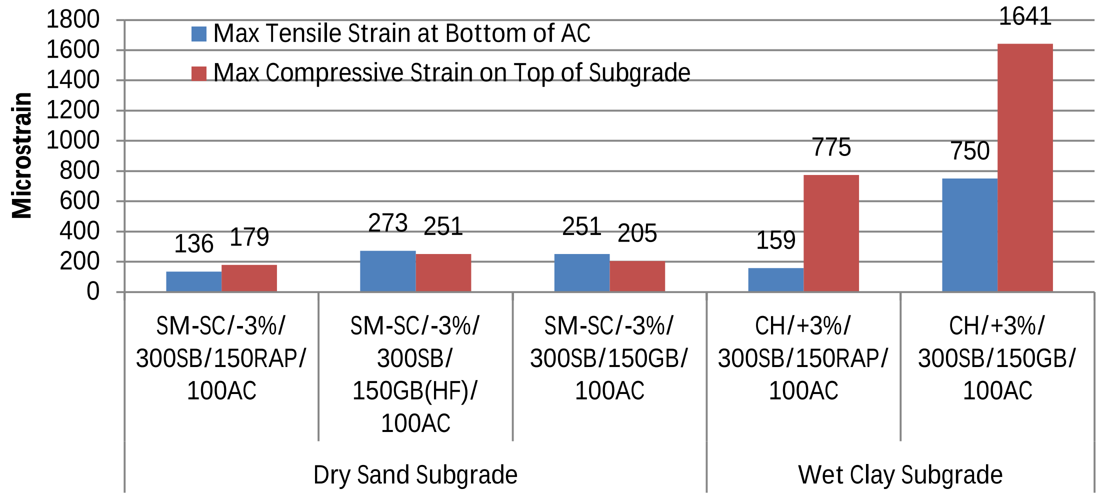


Figure 5.19 Arterial Structural Strain Comparisons with Different Granular Materials

Table 5.15 Arterial Shear Strain Comparisons across Different Granular Materials

Modeled Structure	Maximum Shear by Layer			
	AC	**Granular Layer**	**Sub-Base/ Drainage**	**Subgrade**
Dry Sand Subgrade				
SM-SC/-3%/300SB/150RAP/100AC	309	375	335	140
SM-SC/-3%/300SB/150GB(HF)/100AC	406	597	443	200
SM-SC/-3%/300SB/150GB/100AC	400	585	394	162
Wet Clay Subgrade				
CH/+3%/300SB/150RAP/100AC	400	507	818	798
CH/+3%/300SB/150GB/100AC	793	1950	1960	1559

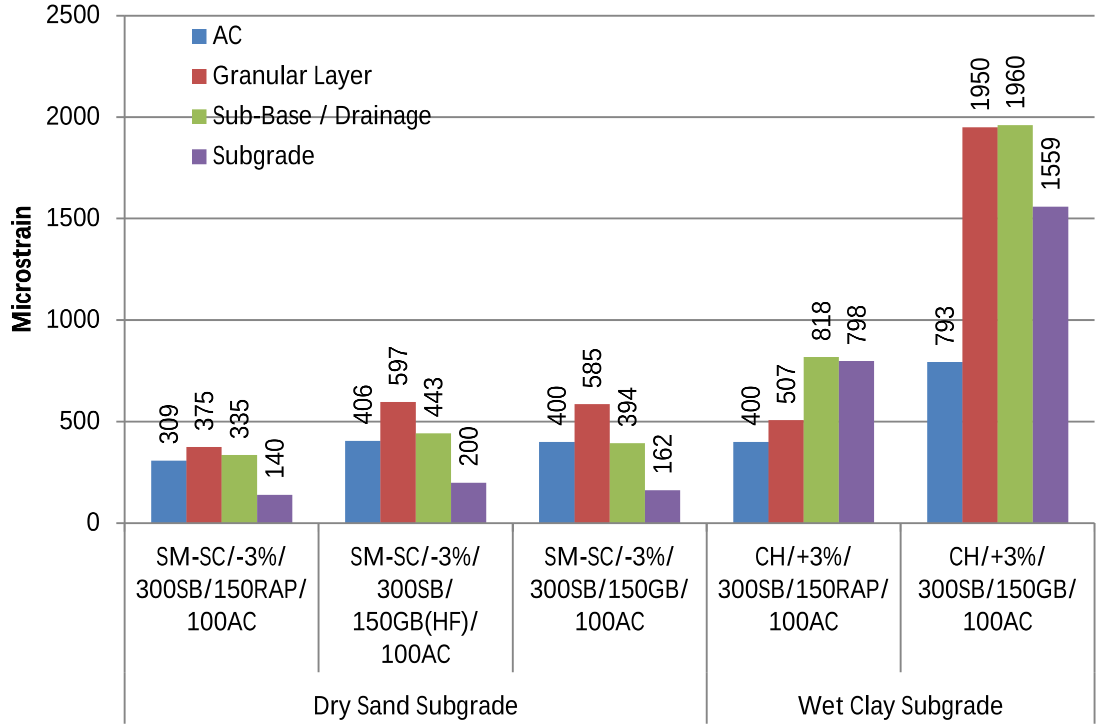


Figure 5.20 Arterial Shear Strain Comparisons across Different Granular Materials

5.2.6 Drainage Layer Sensitivity

As shown in Section 5.2.4, adding an extra 150mm of granular base had little effect on the strain model. The City of Saskatoon has also constructed sections of roadway with a drainage layer to reduce the moisture content in the granular layers. In order to determine the effectiveness of the drainage layer, the local structure was modeled with a 225mm drainage layer and the arterial structure was modeled with a 250mm drainage layer. In the case of the local structure, the granular layer above the structure was changed from 225mm to 150mm when drainage was present to reflect the additional structural capacity of the drainage layer and to minimize the increased aggregate use due to the drainage layer. In the arterial structure, the drainage layer replaced an equivalent thickness of sub-base with the remainder of the structure consisting of granular base. Therefore, the drainage structure was the same total aggregate thickness as the baseline arterial structure.

5.2.6.1 Local Roadway Structure

Substituting the local roadway drainage structure on a wet subgrade showed a significant improvement in the modeled strain responses as shown in Table 5.16 and illustrated in Figure 5.21, dropping the maximum compressive strain on the subgrade by 91.1 percent. The tensile strains at the bottom of the AC also showed a reduction of 83.2 percent when a drainage layer is present.

Table 5.16 Local Structural Strain Comparisons with and without Drainage

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Wet Subgrade without Drainage CH/+3%/225GB/45AC	1440	4617
Wet Subgrade with Drainage CH/+3%/225CR/150GB/45AC	242	409

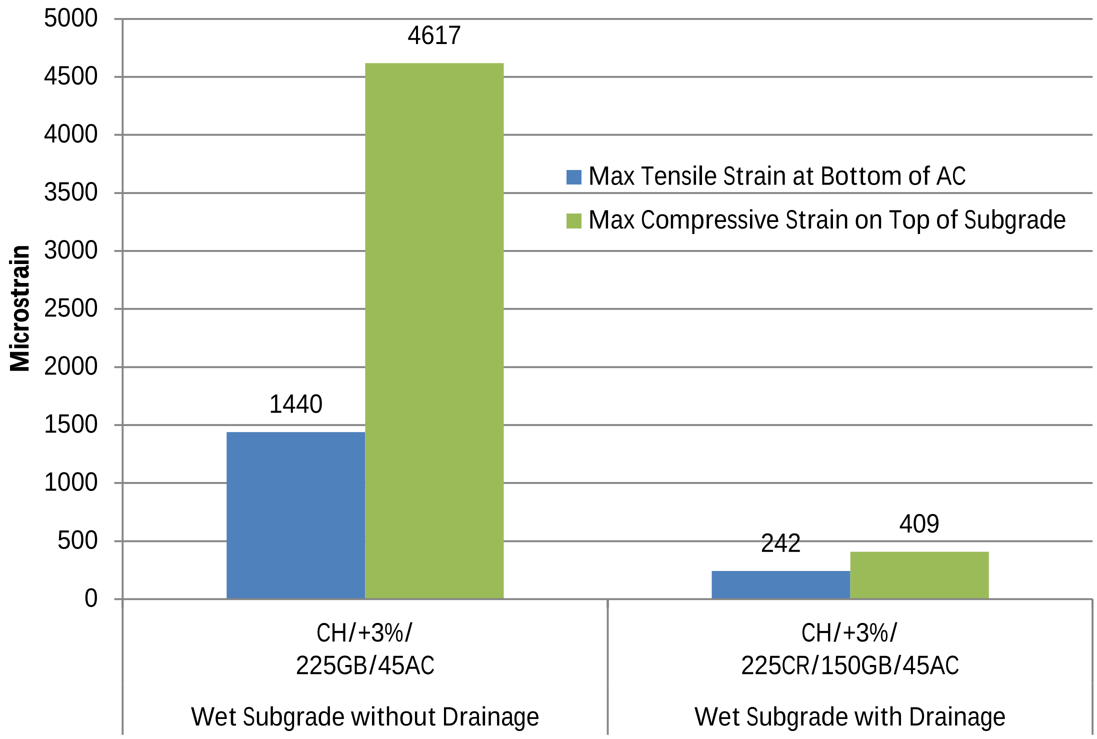


Figure 5.21 Local Structural Strain Comparisons with and without Drainage

The significant reductions seen through the conventional parameters are confirmed when the maximum shear strains in each layer are determined with the FEM as shown in Table 5.17 and illustrated in Figure 5.22. The drainage layer reduces the maximum shear strain in the AC, granular layer and subgrade by 65.8 percent, 80.6 percent and 92.4 percent, respectively.

Table 5.17 Local Shear Strain Comparison with and without Drainage

Modeled Structure	Maximum Shear by Layer		
	AC	**Granular Layer**	**Subgrade**
Wet Subgrade without Drainage CH/+3%/225GB/45AC	1697	5044	4767
Wet Subgrade with Drainage CH/+3%/225CR/150GB/45AC	581	979	363

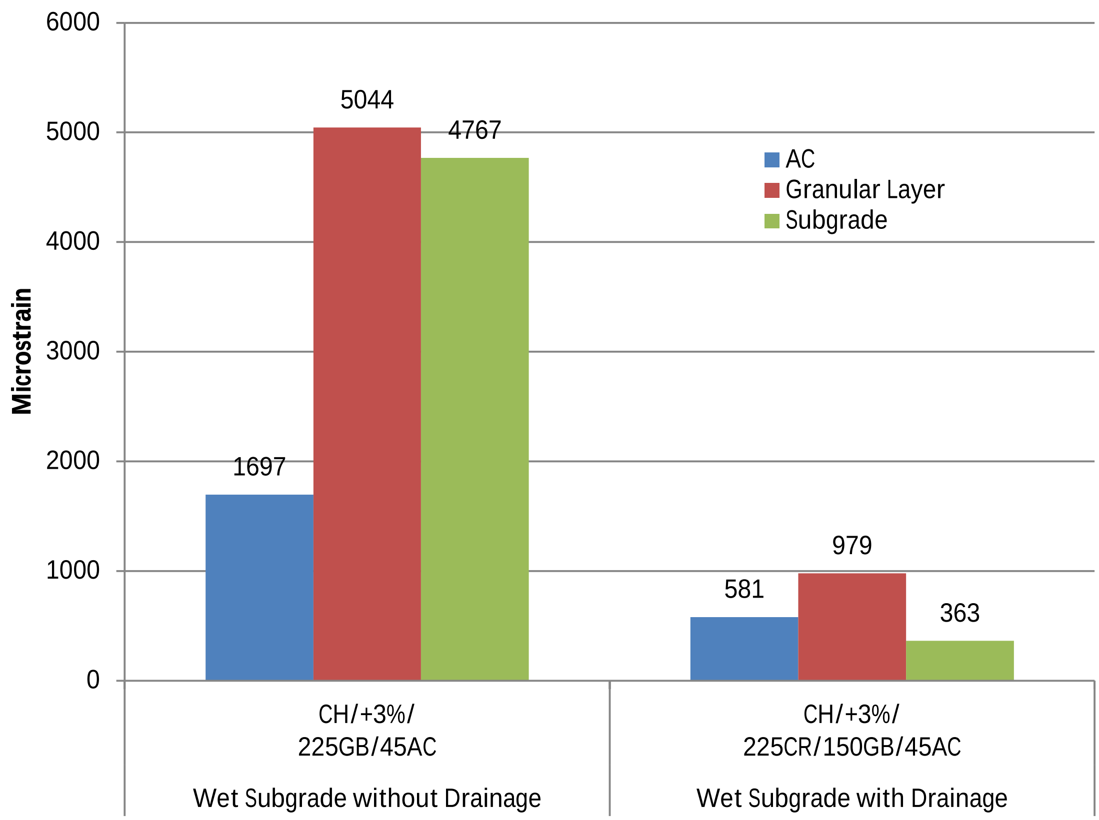
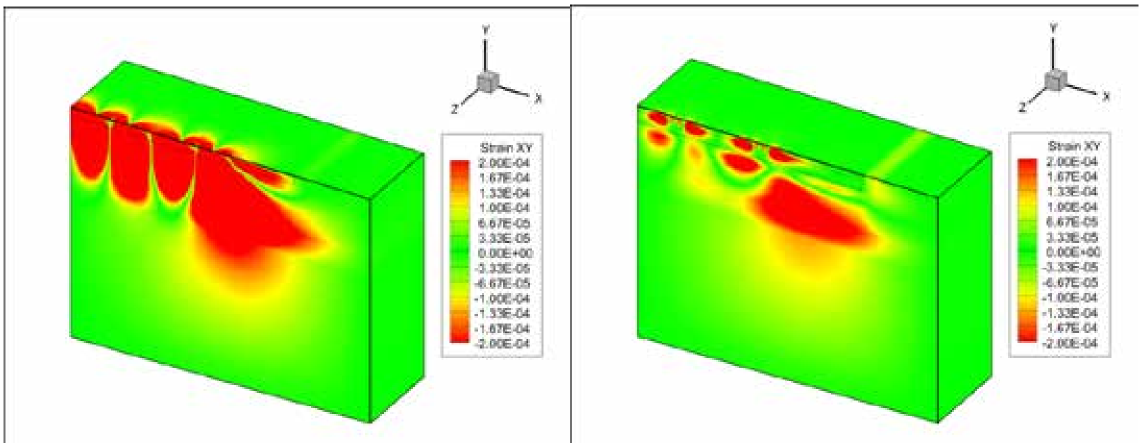


Figure 5.22 Local Shear Strain Comparison with and without Drainage

Illustrating the difference in shear strains is Figure 5.23 which compares, side by side, the wet local structure with and without a drainage layer. The left profile shows the wet structure on clay without any drainage while the right includes a drainage layer. The drainage layer is seen to dissipate the stress so that strains are reduced in the subgrade and throughout the roadway structure.



a) Wet Subgrade without Drainage **b) Wet Structure with Drainage**

Figure 5.23 Standard versus Drainage Local Roadway Shear Strain Comparison

5.2.6.2 Arterial Roadway Structure

Substituting the arterial roadway drainage structure on a wet subgrade showed a significant improvement in the modeled strain responses as shown in Table 5.18 and illustrated in Figure 5.24, dropping the maximum compressive strain on the top of the subgrade by 83.7 percent. Introducing RAP as the base aggregate further reduced the vertical strain on the subgrade by16.7 percent from the conventional base aggregate with drainage. Similarly, the drainage layer reduced the tensile strain at the bottom of the AC by 71.5 percent with conventional granular base. The RAP as the base aggregate further reduced the tensile strain by 50.4 percent.

When the maximum shear strains by layer are compared, as shown in Table 5.19 and illustrated in Figure 5.25, the drainage layer structures reduce the shear strains in the AC, granular, sub-base / drainage, and subgrade by 52.4 percent, 71.3 percent, 94.5 percent and 82.9

percent, respectively. Further, by substituting RAP for conventional granular base, the maximum shears in the AC, granular, sub-base / drainage and subgrade layers are reduced by an additional 25.2 percent, 44.5 percent, 0.9 percent and 9.4 percent, respectively. The RAP reduces the shear strains near the surface while the drainage layer reduces the shear strains on the lower layers.

Table 5.18 Arterial Structural Strain Comparisons with and without Drainage

Modeled Structure	Max Tensile Strain at Bottom of AC	Max Compressive Strain on Top of Subgrade
Wet Subgrade without Drainage		
CH/+3%/300SB/150GB/100AC	750	1641
Wet Subgrade with Drainage		
CH/+3%/250CR/200GB/100AC	214	268
CH/+3%/250CR/200RAP/100AC	106	223

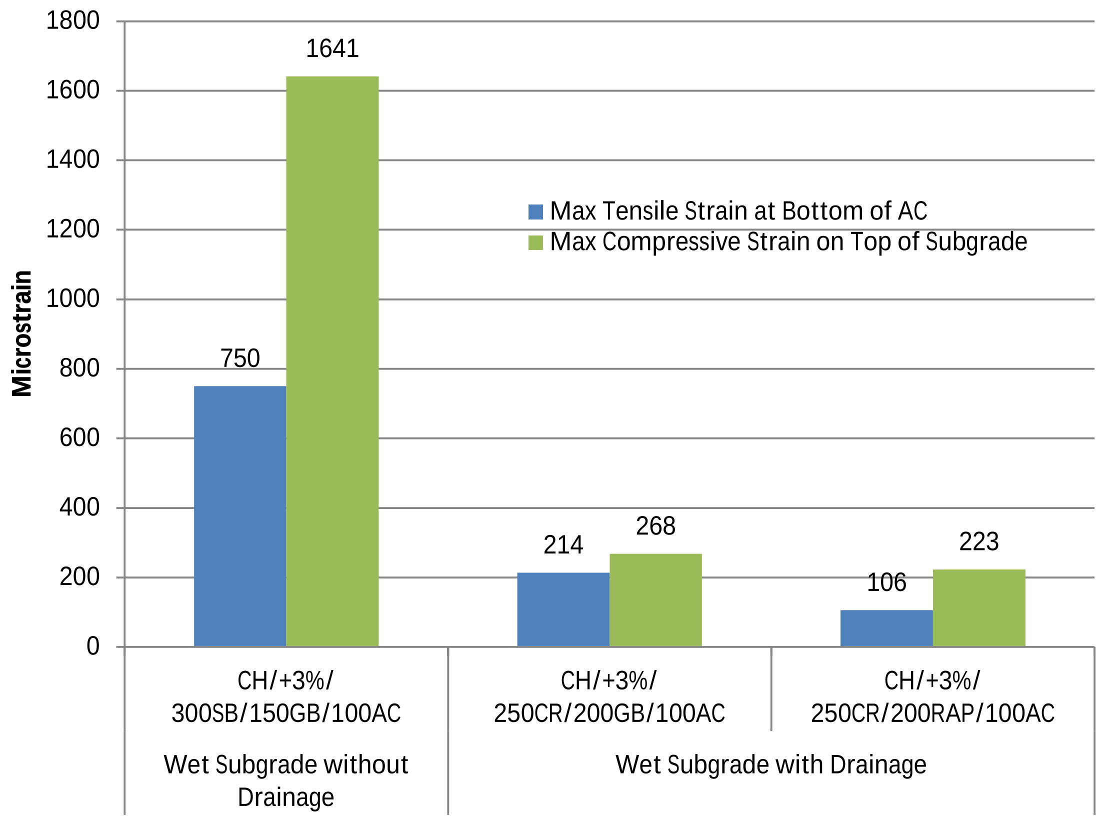


Figure 5.24 Arterial Structural Strain Comparisons with and without Drainage

Table 5.19 Arterial Shear Strain Comparison with and without Drainage

Modeled Structure	Maximum Shear by Layer			
	AC	Granular Layer	Sub-Base/ Drainage	Subgrade
Wet Subgrade without Drainage				
CH/+3%/300SB/150GB/100AC	793	1950	1960	1559
Wet Subgrade with Drainage				
CH/+3%/250CR/200GB/100AC	377	560	108	266
CH/+3%/250CR/200RAP/100AC	282	311	107	241

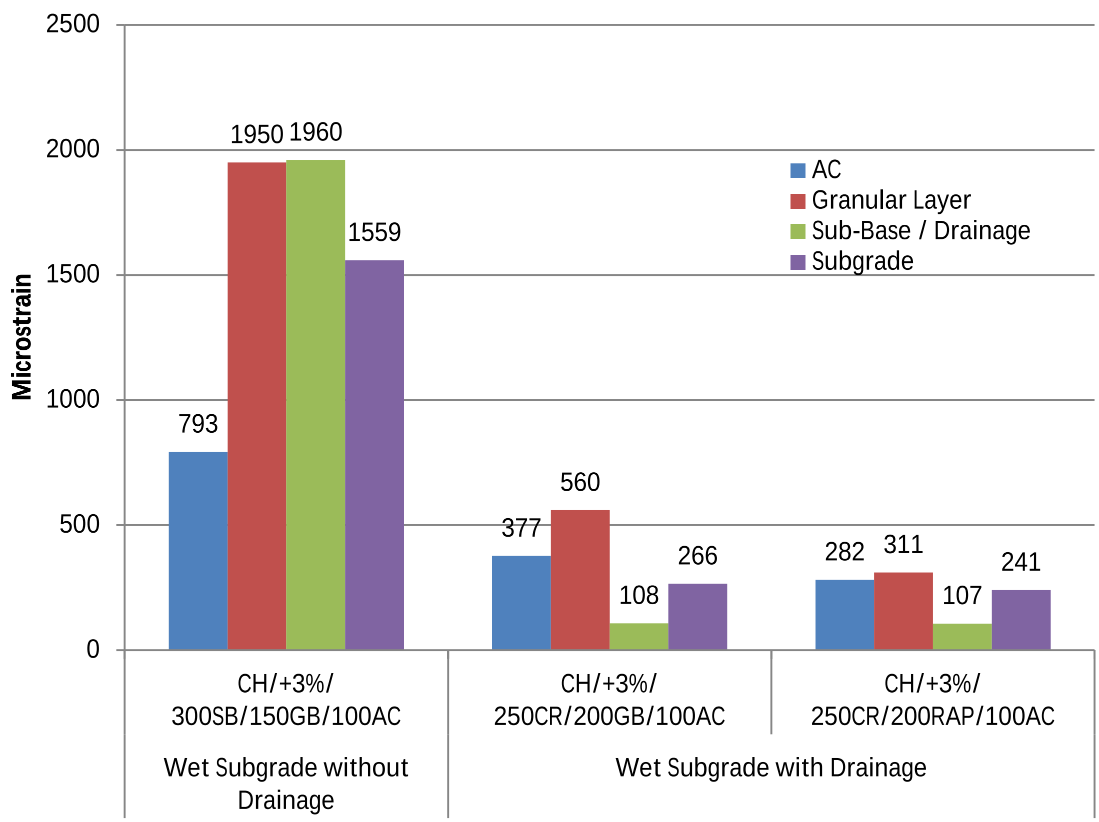


Figure 5.25 Arterial Shear Strain Comparison with and without Drainage

5.3 Chapter Summary

Roadways are designed to protect the subgrade from strains experienced through the repeated traffic loading. When evaluating the variables experienced in the City of Saskatoon field state conditions through a FEM analysis it is evident that the existing cross sections currently used by the City of Saskatoon do not sufficiently protect the subgrade. As well, the subgrade is not prepared to the same extent that would typically happen in a highways project and is required to follow the modified shell curve design. The traffic counts used to devise the thicknesses, especially the local roadways, leave the roadways open to damage through critical loading.

When evaluating the local and arterial roadways based on the mechanical characteristics of the construction materials by changing the subgrade type and moisture level, the standard structures failed when the subgrade became wet. As well, the typical methodology of thickening the granular structure by 150mm in wet conditions has little to no benefit to the subgrade.

Substituting RAP for the granular layer decreased the tensile and compressive strains on the roadway structure by up to 33 percent while also decreasing the shear strains by up to 50 percent. The effect was more noticeable in the local structures as the granular layers were much thinner than the arterial structure granular layers.

Substituting a drainage layer decreased the maximum strains by up to 95 percent. This stress dissipation decreases the strains on the subgrade to levels lower than experienced in a dry subgrade situation, indicating long term benefits to the drainage layer.

CHAPTER 6 FIELD STRUCTURAL PERFORMANCE VALIDATION

Given the primary response relationships shown in Chapter 5, the structures constructed in the test sections all included a drainage layer to minimize the risk associated with a wet subgrade.

In 2009 and 2010, the City of Saskatoon constructed seven test sections with various granular materials in locations where the standard construction methods had been determined to be insufficient. Each of the test sections was in an area where the existing, relatively new roadway had structurally failed. The test sections constructed were Marquis Drive, 115th Street, 8th Street, Kenderdine Road and Field House Road in 2009 while Kenderdine Road, Wilkenson Crescent and Adolph Way were constructed in 2010. Photo logs of the test section construction are found in Appendix C.

6.1 Test Site Preconstruction Conditions

Each of the test sections considered in this research failed prior to the design life of the roadway. The roads were designed to require their first treatment after fifteen years. Table 6.1 indicates the year of construction of each of the test section locations.

Table 6.1 Test Section Original Years of Construction

Test Section	Year of Construction	Year Reconstructed
Marquis Drive	1986	2009
115th Street	1987	2009
Kenderdine Road # 1	1989	2009
8th Street	1984	2009
Field House Road	Data Not Available	2009
Kenderdine Road #2	1990	2010
Wilkenson Crescent	1985	2010
Adolph Way	1984	2010

While the design life was fifteen years, the roadways were not constructed with the expectation of the first treatment being a reconstruction, rather a preservation or restoration treatment such as a resurfacing. However, each of the roadways had structurally failed prior to the year they were reconstructed. Funding shortfalls required the roadways to be patched together until a time when sufficient funds were available for a reconstruction. When the HWD testing was conducted on each segment, structural weaknesses were identified in areas where there was no visual evidence of structural failure. However, once failure started to show, the failures became quite large and obvious. Figure 6.1 indicates a typical structural failure on Marquis Drive. Moisture in the subgrade and granular structure has weakened the roadway so that it will not support the traffic loading. As the loading exceeds the capacity of the roadway, the asphalt structure deforms and creates "alligator cracks" that eventually break entirely out of the roadway forming these failed sections.

Figure 6.1 Structural Failure on Marquis Drive

6.1.1 Structure Removal and Recovery

In order to address substructure moisture issues, the City of Saskatoon decided to construct drainage layers tied into the storm sewer system. This drainage layer required the entire road structure to be removed and replaced with a new structure.

In all test sections the roadway asphalt structure was rotomixed, as shown in Figure 6.2, and stockpiled either on site, as shown in Figure 6.3, or brought to the City of Saskatoon material handling site, depending on available room and distance to the material handling site. This material consisted of the existing 50mm to 100mm of asphalt and the top 50mm to 100mm of the granular layer. The remaining granular and subgrade material was hauled away to a stockpile site to be reused as general fill in other public works operations.

Figure 6.2 Cold In-Place Rotomixing Operation on 115th Street

Figure 6.3 Stockpiling of Recycled Granular on Marquis Drive

6.1.2 Test Section Structure Construction

Once the excavation was complete, a non-woven geotextile was placed on the subgrade to act as a separation layer. A biaxial geogrid was then placed on top of the geotextile to provide a working platform to start constructing the roadway structure. In some cases, the subgrade was too wet to construct on and needed to be over excavated and replaced with sand to allow construction to continue.

Samples of the subgrade were taken and tested to determine Atterberg limits and gradation. The *in situ* subgrade materials ranged from low plastic clays to high plastic clays.

The drainage aggregate was hauled either from Nicolson yard, just east of the City of Saskatoon, or the contractors material handling site, and placed on the geogrid along with edge drains connected to the storm sewer system. All the test sections except the 115th Street location used crushed portland cement concrete as a drainage layer. The 115th Street test section used pit crushed rock as the drainage aggregate. The drainage aggregate was then covered with a woven geotextile, shown in Figure 6.4, to separate the granular aggregate from the drainage layer, preventing a movement of fine material into the drainage layer.

Once the structural aggregate layers were complete, an asphalt emulsion was applied to the surface, blended into the top 100mm of the aggregate as shown in Figure 6.5, and then compacted to optimum density. This blend provided a surface that was moisture resistant and ready for asphalt surfacing. The surface was then paved and ready for traffic as shown in Figure 6.6.

Figure 6.4 Recycled Aggregate Layer and Geotextiles on Wilkinson Crescent

Figure 6.5 Blending of Emulsion into Aggregate Surface on Kenderdine

Figure 6.6 Paved Surface on 115th Street Test Section

6.2 Heavy Weight Deflectometer Structural Testing

One of the tools used by the City of Saskatoon asset management system is heavy weight deflectometer testing. With these tests, various standardized weights are dropped on the roadway and the deflection response is measured. Each road class has measured deflections zones that classify the roadway structural response as either good, fair, poor or very poor as identified by Table 6.2.

Table 6.2 COS HWD Structural Response Classifications

Rating	Local	Collector	Arterial	Expressway
Good	<1.0	<0.65	<0.5	<0.5
Fair	1-1.75	0.65-1.00	0.5-0.75	0.5-0.75
Poor	1.75-2.5	1.00-1.5	0.75-1.0	0.75-1.0
Very Poor	>2.5	>1.5	>1.0	>1.0

Given this response table, the various roadway test sections were evaluated using their primary weight to determine if the post construction response was within the required end classification of good. Table 6.3 and Figure 6.7 indicate how the local test sections matched up to their required testing response while Table 6.4 and Figure 6.8 indicate the test results for the arterial test locations.

Table 6.3 HWD Peak Surface Deflection Test Results For Local Test Sections

Test Section		Average	5th Percentile	95th Percentile	Coefficient of Variance (%)
Adolph	Pre	3.38	2.14	4.00	26%
	Post	1.24	1.12	1.43	11%
	% Reduction	63%	48%	64%	58%
Wilkinson	Pre	2.25	1.89	2.61	38%
	Post	1.26	1.01	1.51	14%
	% Reduction	44%	47%	42%	63%

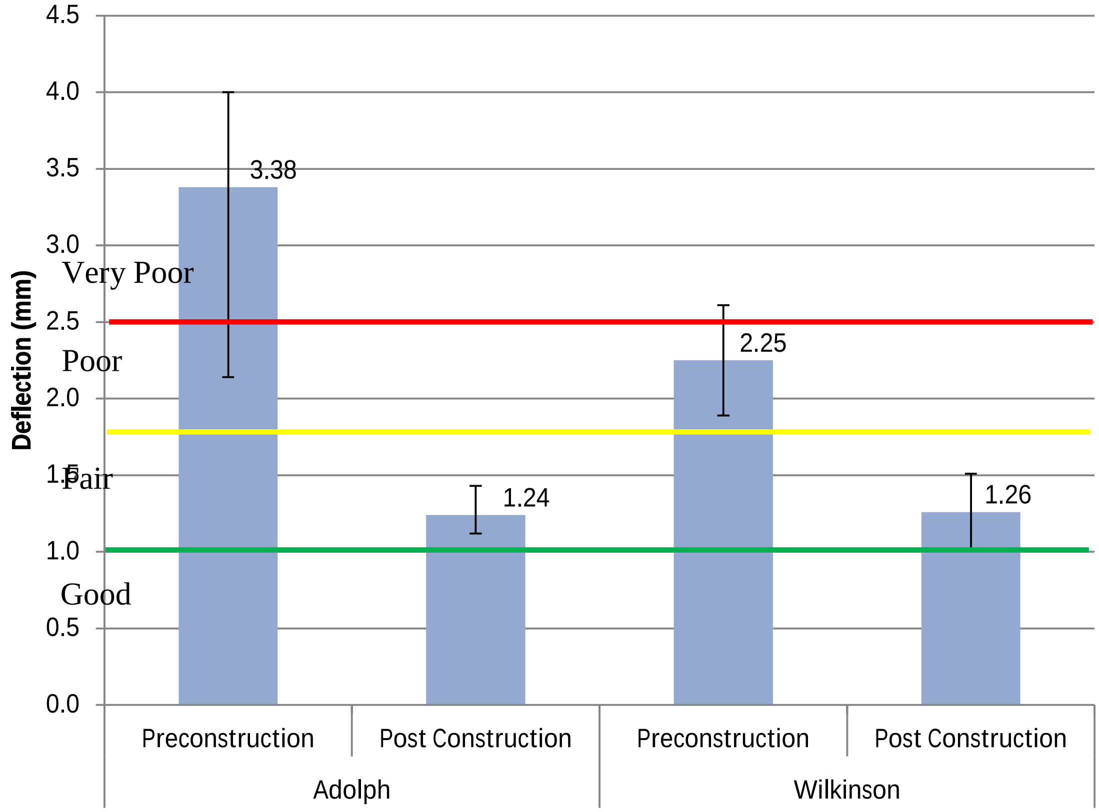


Figure 6.7 HWD Peak Surface Deflection Test Results For Local Test Sections

The local structures, as constructed, did not meet the good rating in the structural response standards of the City of Saskatoon. During construction, the subgrade content was found to be higher than optimum moisture and close to the liquid limit in portions of the test sections. The high moisture contents in the constructed test sections caused the subgrade to have less structural capacity than the model predicted. However, the structures showed a significant improvement to the preconstruction conditions with a deflection at primary weights of 63% less on Adolph and 44% less on Wilkinson. As well, the coefficient of variance was decreased for these two test sections by 58% for Adolph and 63% for Wilkinson. The reduced variance indicates that the structural response is less dependent on subgrade moisture conditions with the drainage layers installed in the test sections.

Table 6.4 HWD Peak Surface Deflection Test Results For Arterial Test Sections

Test Section		Average	5th Percentile	95th Percentile	Coefficient of Variance (%)
Marquis	Pre	2.19	1.28	3.17	50%
	Post	0.55	0.50	0.60	10%
	% Reduction	75%	61%	81%	79%
Kenderdine #1	Pre	1.50	0.78	2.49	34%
	Post	0.47	0.35	0.6	17%
	% Reduction	69%	55%	76%	50%
115th St	Pre	1.03	0.69	1.43	29%
	Post	0.57	0.53	0.62	9%
	% Reduction	45%	23%	57%	69%
8th Street	Pre	0.92	0.60	1.29	24%
	Post	0.44	0.24	0.64	32%
	% Reduction	52%	60%	50%	-33%
Field House Road	Pre	1.49	1.45	1.53	34%
	Post	0.4	0.3	0.5	4%
	% Reduction	73%	79%	67%	89%
Kenderdine #2	Pre	1.46	0.84	2.45	21%
	Post	0.72	0.6	1.02	26%
	% Reduction	51%	29%	58%	-23%

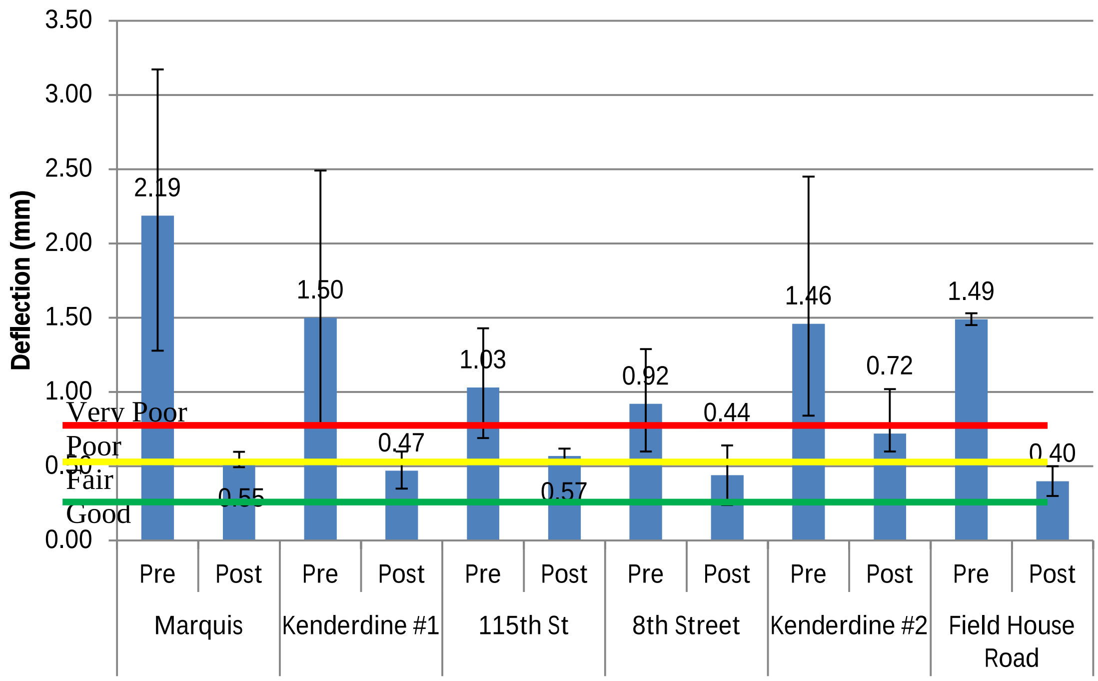


Figure 6.8 HWD Peak Surfaced Deflection Test Results For Arterial Test Sections

As shown in Figure 6.8, the arterial test section post construction surface deflection results were all rated as either good or fair. Prior to the reconstructions each of these arterial test sections were rated as very poor. Deflections due to primary loading were reduced between 45 percent and 75 percent. The use of the drainage layer in these poor subgrade areas showed a significant improvement in the structural response of the roadway.

6.3 Test Section Structural Validation

The FEM software has the ability, by determining the strain characteristics, to predict the deflections on a cross section given a loading scheme. By modelling the primary weight response, comparisons can be made between field state conditions and constructed roadways with the projected deflections.

All test sections were constructed in roadways that had failed structurally and required reconstruction to be placed back in service. Construction of the test sections showed variability in the existing granular material as well as variability in the subgrade type and moisture content. While construction of the test sections was attempted to be uniform, site conditions and field decisions by construction staff added variations to the design cross sections such as overexcavated zones, additional geosynthetics and localized granular reinforcement. In order to compare the model to the pre and post construction conditions of the test sections, the most uniform test section was selected. Kenderdine Road #1 was constructed with a consistent cross section and had the most consistent subgrade throughout the test area.

As Kenderdine Road #1 was originally constructed to the standard structure for an arterial roadway but the subgrade ended up being wet, the preconstruction HWD testing should line up with the modeled deflection response in section 5.2.2. The *in-situ* subgrade used within the model was from the Kenderdine site location. As shown in Table 6.5 and illustrated in Figure 6.9, the baseline modeled cross section was within the coefficient of variance of the average preconstruction deflection of the test section. As well, the constructed test section, identified as the post construction HWD test, matched well with the modeled response of the drainage structure #2 which was the design cross section for the Kenderdine test section.

Table 6.5 HWD Comparison Between Theoretical Model Prediction And Actual Deflections

Deflection	Kenderdine #1 Pre Construction	Modeled Baseline	Kenderdine #1 Post Construction	Modeled Drainage (RAP)
Average (mm)	1.50	1.87	0.47	0.45
5th Percentile (mm)	0.78	-	0.35	-
95th Percentile (mm)	2.49	-	0.60	-

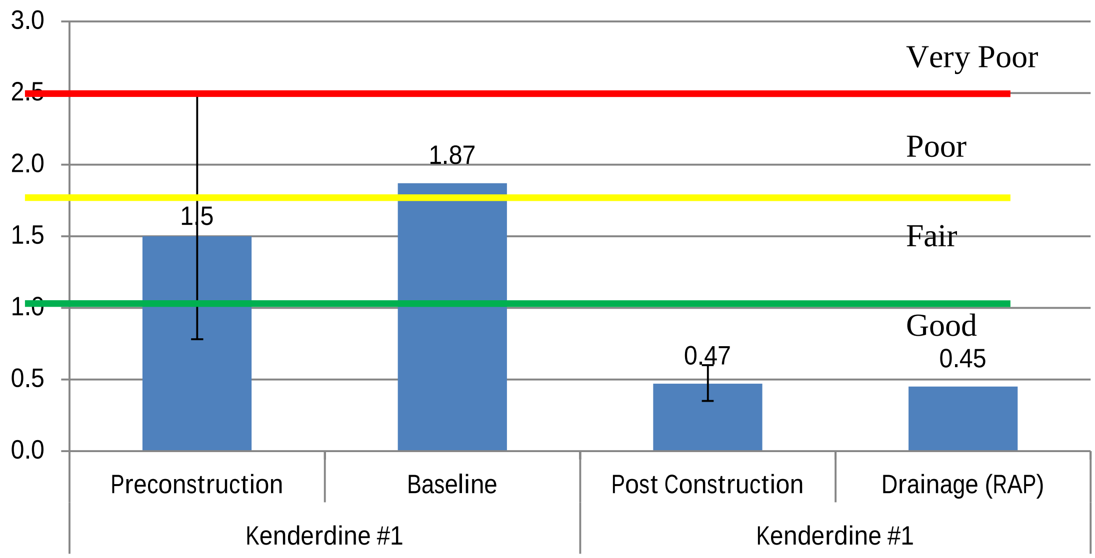


Figure 6.9 HWD Comparison Between Theoretical Model Prediction And Actual Deflections

6.4 Chapter Summary

All post construction HWD testing showed a significant improvement in the primary weight deflection responses. While the local test sections did not improve to the good level, as identified by the City of Saskatoon asset management system, the drainage layer did bring the local structures into the fair category and decrease the deflection by 44 to 63 percent.

The arterial test results all showed structures in the good category with deflection results decreasing by 45 to 75 percent in the arterial test sections. As the constructed structures were no thicker than the standard structures used by the City of Saskatoon, the construction of a drainage layer is integral in improving the structural responses on wet subgrades. When comparing the field structural response to the modeled response a very close correlation is evident. As the field state conditions and construction materials matched the modeled inputs in this case, the correlation shows the strength of the model. While the drainage layers are initially performing as expected and within deflection requirements of the City of Saskatoon, the long term performance of the drains is not proven and a maintenance schedule for the drain pipes is required to retain the drainage characteristics.

CHAPTER 7 END PRODUCT SUSTAINABILITY ANALYSIS

In order for any growth to be deemed sustainable it must be economically sustainable as well. This applies to how roadways are built as well. Each of the structures identified in Figure 5.4 and Figure 5.5 have a defined capital cost to construct as well as a defined service life that can be evaluated against historic results through previous research. In order to determine if using recycled materials is sustainable, the environmental benefits are also a key element, as identified in Figure 7.1. A simple way of determining the environmental benefits is to evaluate the energy usage for the construction of the roadway. The social benefits of the recycled material are represented in stronger roadways as identified in chapters 5 and 6.

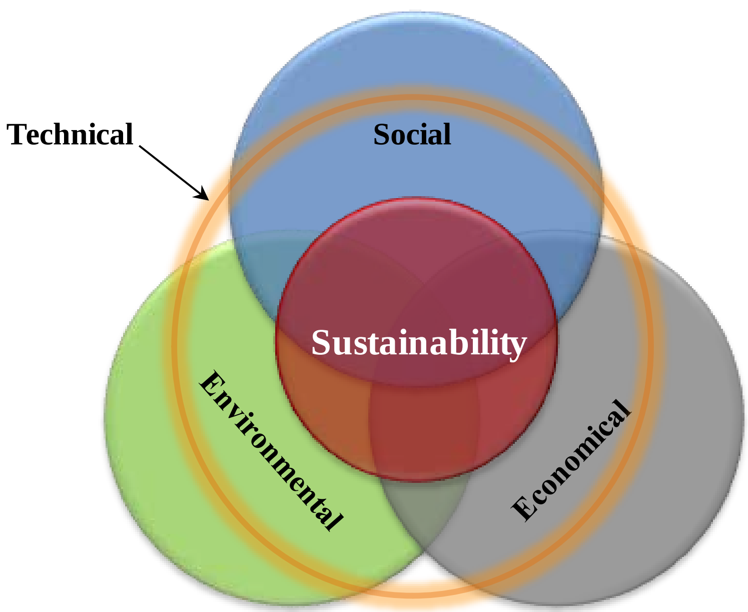


Figure 7.1 Sustainability Diagram

7.1 Capital Cost Evaluation

Construction costs of the design cross sections evaluated in chapter 5 vary based on whether recycled aggregates are used within the granular layers or not. Unit costs and structure thicknesses are identified in Appendix D.

The baseline structure was evaluated with RAP used as the granular base substitute and is referred to as baseline (RAP). The drainage structures are similarly defined as drainage structure and drainage structure (RAP). As shown in Table 7.1 and illustrated in Figure 7.2, the capital cost for the drainage layer designs for local roadways designed in chapter 5 have a higher capital cost than the standard City of Saskatoon roadway structures while the structures with RAP as a granular base aggregate have a lower capital construction cost. The increase in cost of the drainage structure can be attributed to the extra cost of geosynthetics installed to drain the system and to maintain the separation between the drainage aggregate and the subgrade as well as the aggregate base. As the standard methodology used in wet subgrades is to thicken the granular base, evaluating that cost compared to the drainage structure costs is more applicable. The drainage structure is 8 percent more expensive than the thickened structure while substituting RAP as the granular layer for the drainage structure costs 1 percent less than the standard thickened structure.

Table 7.1 Capital Construction Cost of Modeled Local Structures

Structure	Capital Cost per Square Meter
Baseline	$ 49.56
Baseline (RAP)	$ 42.13
Thickened	$ 58.41
Drainage Structure	$ 62.93
Drainage Structure (RAP)	$ 57.98

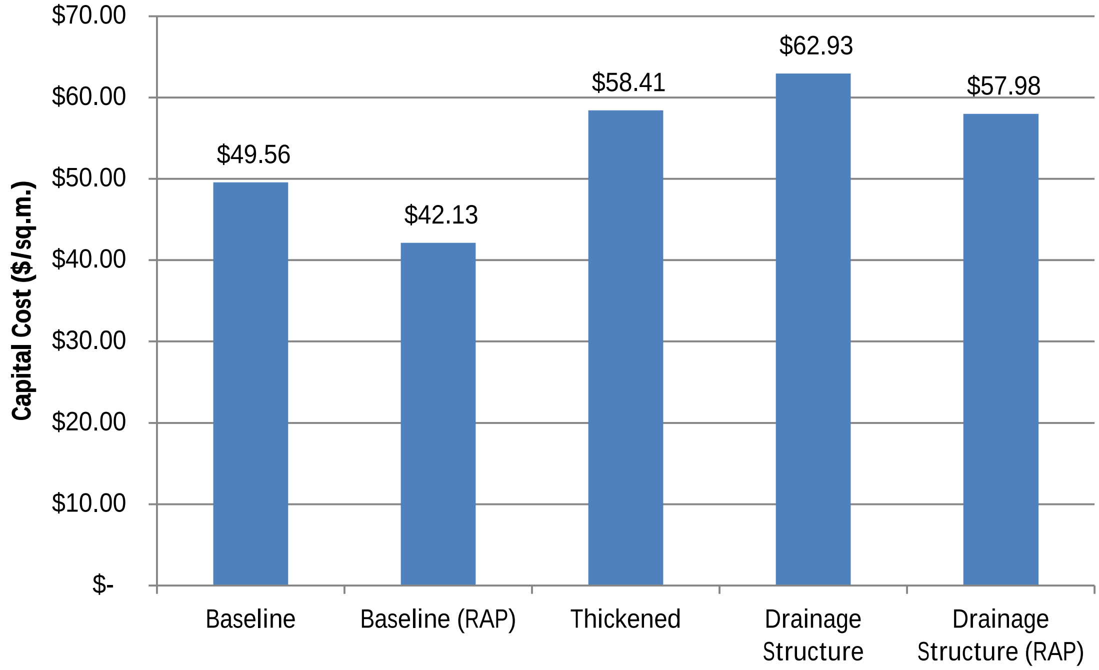


Figure 7.2 Capital Construction Cost of Modeled Local Structures

When evaluating the capital cost of the arterial structures shown in chapter 5, the aggregate costs are more significant. Since the drainage structure is the same thickness as the conventional arterial structure it is only 10 percent more expensive to build, entirely due to the required geosynthetics. As shown in Table 7.2 and illustrated in Figure 7.3, the structures constructed with recycled materials have a lower capital cost than their equivalent structure constructed with pit derived aggregates.

Table 7.2 Capital Construction Cost of Modeled Arterial Structures

Structure	Capital Cost per Square Meter
Baseline	$ 94.81
Baseline (RAP)	$ 89.86
Thickened	$ 103.66
Drainage Structure	$ 94.64
Drainage Structure (RAP)	$ 71.24

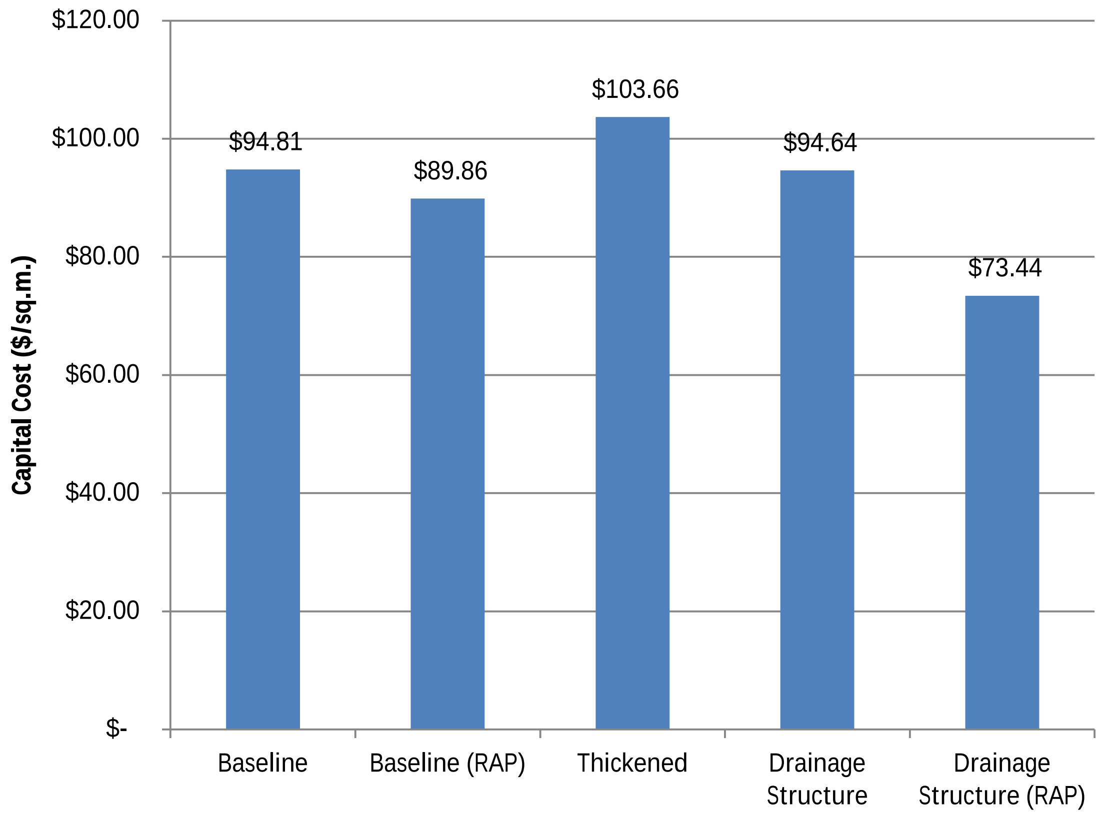


Figure 7.3 Capital Construction Cost of Modeled Arterial Structures

7.2 Service Life Illustration

As an illustrated example of costs over the service life of the roadway, the treatment schedule for roadwork on local roads in poor subgrade neighborhoods until the roadway requires replacement. While structural failure for many of the investigated roads occurred prior to 20 years, budget restraints required an average of 20 years between construction to the reconstruction of the test section roadways. As the deflection data and modeled responses show, when a drainage layer is installed, the structures perform as if they are on a good dry subgrade. The illustration uses an example of microsurfacing the structurally sound roads every 15 year and a resurfacing treatment at the midpoint of the service life of the structurally sound roadway. A discount rate of 3% was used in the illustration to bring all construction costs back to present value. Figure 7.4 illustrates the cost per square meter for five different initial structures and subsequent roadway treatments over the roadway service life.

Table 7.3 Present Value of Local Roadway Construction and Preservation Treatments

Structure	Treatment	Cost	Year	Present Value
Baseline	Construction	$49.56	0	$49.56
Thickened	Construction	$58.41	0	$58.41
Drainage Structure	Construction	$62.93	0	$62.93
	Microsurfacing	$7.50	15	$4.81
	Microsurfacing	$7.50	30	$3.09
	Resurfacing	$45.00	45	$11.90
	Microsurfacing	$7.50	60	$1.27
	Microsurfacing	$7.50	75	$0.82
Drainage Structure - RAP Base	Construction	$57.98	0	$57.98
	Microsurfacing	$7.50	15	$4.81
	Microsurfacing	$7.50	30	$3.09
	Resurfacing	$45.00	45	$11.90
	Microsurfacing	$7.50	60	$1.27
	Microsurfacing	$7.50	75	$0.82

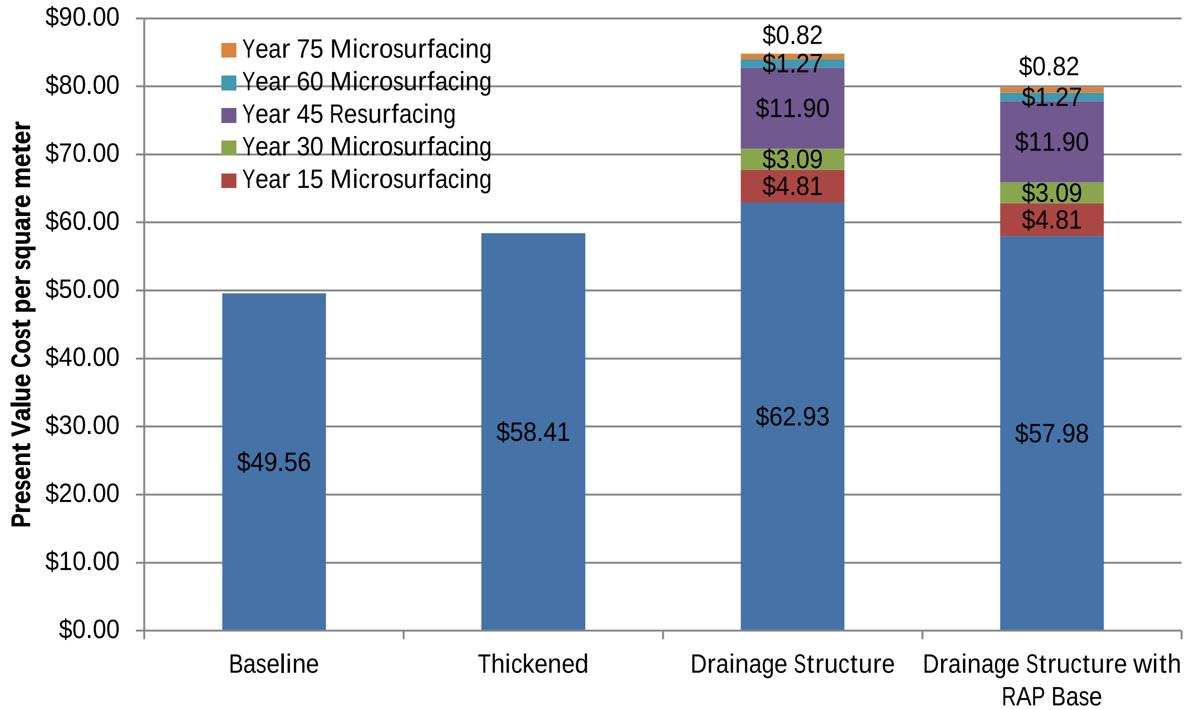


Figure 7.4 Present Value of Local Roadway Construction and Preservation Treatments

Each roadway in the illustration is replaced between 20 and 90 years after construction. When the construction cost and the treatment costs are annualized, the annual costs range from $2.48 per square meter for the baseline example to $0.89 per square meter for the drainage example with a RAP base. The annualized cost and time before replacement is shown in Figure 7.5 and Table 7.4. Adding a drainage structure, which allowed for surface preservation and restoration treatments to occur, reduced the annualized cost of the roadway to less than 40% of the baseline structure.

Table 7.4 Years of Service and Annualized Cost for Sample Treatment Schedule of Local Roadway

Structure	Years of Service	Total Cost	Annualized Cost
Baseline	20	$49.56	$2.48
Thickened	25	$58.41	$2.34
Drainage Structure	90	$84.82	$0.94
Drainage Structure - RAP Base	90	$79.87	$0.89

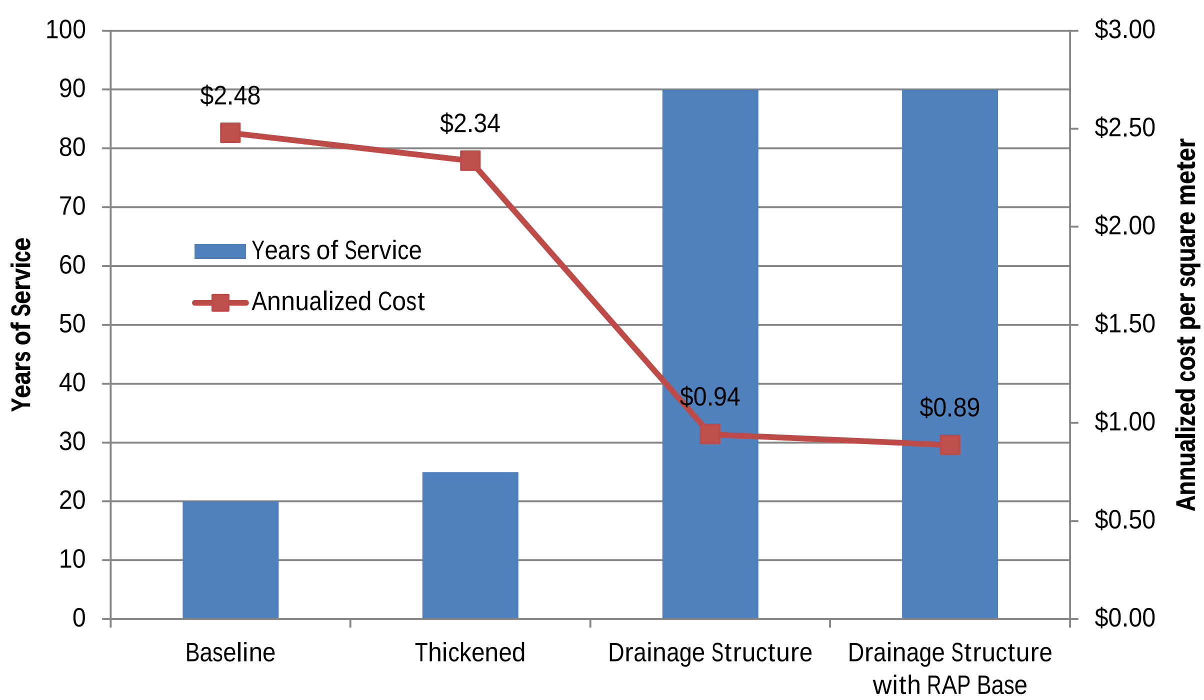


Figure 7.5 Years of Service and Annualized Cost for Sample Treatment Schedule of Local Roadway

7.3 Energy Usage Comparison

A significant portion of the energy usage in constructing a roadway is taken in hauling materials to the job site. When the different options for building a roadway are evaluated based on their energy use, using recycled material within the roadway decreases the amount of energy used. The modeled cross sections of local and arterial roadways were inputted into an energy use model developed at the University of Saskatchewan (Haichert, J., Bajpai, & Berthelot, 2009). Details of the model can be found in Appendix E.

From this energy use model shown in Table 7.5 and illustrated in Figure 7.6, the thicker local structures used 27 percent more energy than the baseline structure. Substituting RAP in the baseline structure reduced the energy usage by 36 percent. When the drainage layer was modeled, the energy use was 23 percent more than the standard structure. Using *in situ* RAP as the base layer with a drainage structure reduced the total energy use by 5 percent from the baseline structure even though the drainage structure includes 67 percent more total aggregate.

As the model was applied to the arterial modeled structures, the energy usage reduction was more evident in the drainage and recycled structures as shown in Table 7.6 and illustrated in Figure 7.7. Substituting RAP for the base aggregate in the baseline structure reduced the energy usage by 12 percent. From the baseline structure, the thickened structure used 13 percent more energy due to the increase in granular structure. However, the drainage structure used 2 percent less energy than the baseline structure, due to a decrease in haul distance when using recycled portland cement concrete as an aggregate source. Including the use of RAP as a portion of the base aggregate reduced the energy usage by 15 percent from the baseline structure for the same amount of granular structure.

Table 7.5 Energy Usage for Local Roadway Construction Options

Structure	Total Energy Consumption (MJ)
Baseline	315,982
Baseline (RAP)	201,698
Thickened	400,214
Drainage Structure	388,150
Drainage Structure (RAP)	305,060

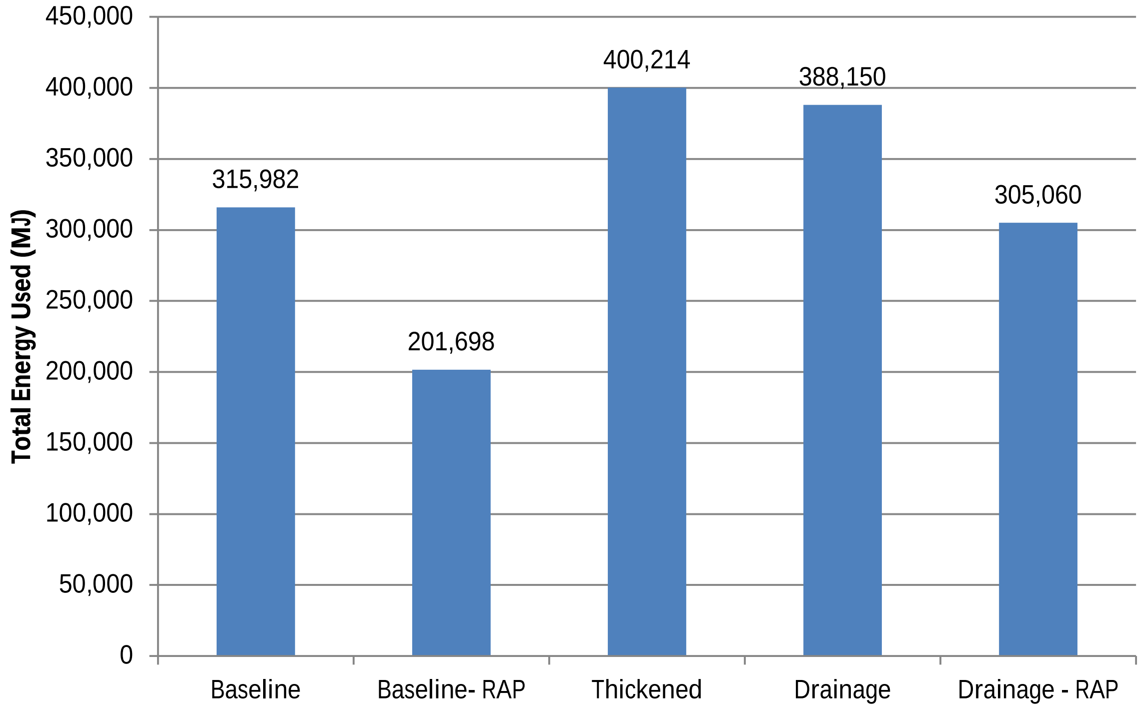


Figure 7.6 Energy Usage for Local Roadway Construction Options

Table 7.6 Energy Usage for Arterial Roadway Construction Options

Structure	Total Energy Consumption (MJ)
Baseline	653,638
Baseline (RAP)	577,449
Thickened	737,870
Drainage Structure	640,234
Drainage Structure (RAP)	558,507

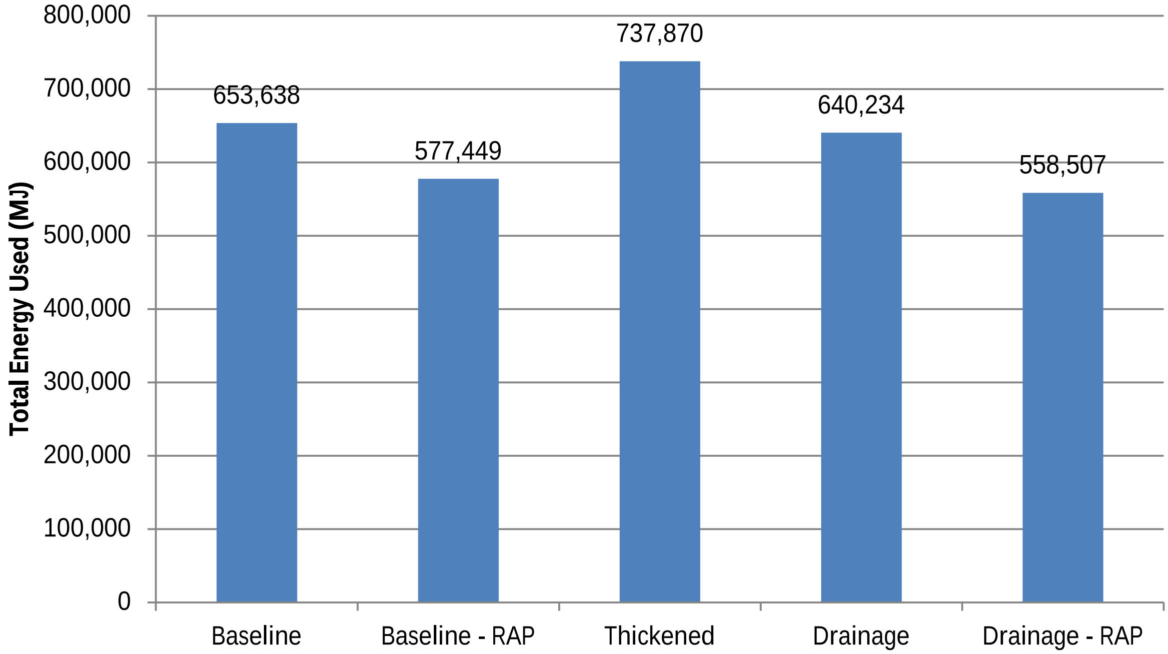


Figure 7.7 Energy Usage for Arterial Roadway Construction Options

7.4 Chapter Summary

While the drainage layers may cost more than the baseline structure, when their costs are compared to the alternative of thickening the structure, for both the local and arterial alternative, the initial capital costs are less. The more recycled aggregate that is used, the less it costs to construct the roadway cross sections.

When this evaluation is extended to the 80 year overall capital costs of the roadway, the drainage structures significantly reduce the cost to construct, preserve and restore the roadway. The cost savings can add up to over 50 percent over the 80 years given standard preservation, restoration and reconstruction costs and scheduling.

Evaluating the design cross sections for energy use to construct also shows a reduction when recycled materials are used for the drainage and granular aggregates. The savings in energy use can translate into lower construction costs and possible energy credits in the future.

CHAPTER 8 DESIGN AND SPECIFICATION RECOMMENDATIONS

When constructing an urban roadway different criteria are used to design the roadway than those used for highway construction. Typical traffic patterns include slow moving, turning traffic, edge loading of heavy traffic including various high ESAL municipal vehicles, and no ability to limit traffic loading in freeze/thaw seasons. As well, a high density of roadways is required in a small area, creating large demands on the localized resources.

Having the increased demand on aggregate resources, as well as a ready supply of recycled aggregates, allows for the inclusion of recycled materials in the roadway design. This, however, is not allowed by standard specifications as shown in Chapter 2. Therefore various changes to the specifications need to be in place to properly design with the recycled aggregates.

8.1 Material Specifications

The standard aggregate tests as outlined by the City of Saskatoon are not written with recycled aggregates in mind. While the recycled aggregates do pass some of the requirements of the standard specifications such as gradation, other tests were not designed for aggregates that have residual cohesion from asphalt or portland cement concrete. When the materials are evaluated for their mechanical characteristics the recycled materials performed better than the conventional aggregates.

Within the aggregate sections of the City of Saskatoon specifications, wording should be added to include the use of recycled portland cement concrete and asphalt aggregate as a substitute for conventional base aggregate provided it meets gradation and fracture requirements. Wording should also be added to include the use of recycled portland cement concrete aggregate as a substitute for conventional crushed rock provided it meets gradation and fracture requirements.

The recycled portland cement concrete and asphalt aggregates will not pass conventional CBR aggregate strength testing. Therefore, aggregate specifications need to include an element of characterizing the mechanistic natures of the aggregates. Other agencies have started using the resilient modulus for designing with MEPDG and have had some success integrating recycled aggregates into the design process. As urban roadways experience a dynamic range of loading frequencies, using a dynamic modulus is better suited for understanding how the roadway materials will perform. The dynamic modulus, as shown in Chapter 3, gives a good indication of the structural capacity of the aggregates. Setting a minimum dynamic modulus and other mechanistic characteristics, as outlined in Table 8.1, for the aggregates along with the gradation specification will characterize the aggregate for its specific structural capacity as well as its moisture characteristics. The minimum or maximum mechanistic characteristics should be based on field state stress conditions similar to those presented in Chapter 4..

As indicated in Chapter 4 the OGBC gradation for both the recycled portland cement concrete and asphalt produced favourable results across all test parameters. Creating a gradation curve allowing for this type of material would allow for the specification of a high quality base that is less prone to moisture damage and provides a higher structural capacity than typical granular base graded aggregate.

Table 8.1 Recommended Mechanistic Characteristics

Mechanistic Property	Subbase Material	Well Graded Base Material	Open Graded Base / Drainage Material
Dynamic Modulus (MPa)	200 (minimum)	250 (minimum)	400 (minimum)
Poisson's Ratio	0.15 to 0.45	0.10 to 0.40	0.15 (minimum)
Phase Angle < 3% AC (degrees)	10 to 25	7 to 15	7 to 15
Phase Angle > 3% AC (degrees)	15 to 35	12 to 25	10 to 20
Radial Microstrain	500 (maximum)	450 (maximum)	300 (maximum)

Using a mechanistic modeling program, like PSI Pave3D™ used within this study, allows the roadway design to be validated post construction. The modeling program, with post construction non destructive testing, allows the validation of the design structure. The roadway designs should be structured so that they meet the City of Saskatoon asset management system deflections for “good” roads as per Table 6.2. Using this validation loop will ensure structurally adequate roads are constructed and allow the designer to create the most economic structure based on current aggregate and AC prices.

8.2 Drainage Layer Design

With a significant portion of the new roadway construction being anticipated in wet areas, the moisture conditions need to be considered in the design method and specifications. As seen in Chapter 6, the typical construction methods are inadequate to deal with soils that are prone to be wet of optimum moisture content. The best solution, as shown by the FEM, was constructing a drainage layer within the roadway structure. The drainage structure reduced the strain on the subgrades by as much as 95 percent in similar soils and traffic loading.

A drainage structure should be constructed with aggregate material that meets the crushed rock gradation as set in the City of Saskatoon specifications. This drainage structure should be used in any roadway that has subgrades that will retain moisture. As shown in Chapters 4 and 5, crushed portland cement concrete can be processed to be a suitable substitute for conventional crushed rock in this application.

Drainage layers will only work as long as they are conveying the moisture out of the granular structure and will require routine maintenance to ensure the drain pipes are not plugged. This is one of the downfalls of using subsurface drainage. As well, the climate in Saskatchewan, with the ground being frozen half the year, results in periods where the drainage layer is frozen, not allowing the moisture to drain from the granular layer.

As shown in Chapter 6, having a 225 mm crushed portland cement concrete drainage layer that also acts as a stress dissipation layer will significantly reduce the strains on all structural layers of the roadway. Based on field test trials, the minimum thickness of the drainage layer is 225 mm in order to provide a good working structure over the softer subgrades during construction.

Having the drainage layer act as a structural stress dissipation layer reduces the aggregate requirement for the roadway structure. The typical arterial structures can then be constructed to the same aggregate thicknesses while the local structure only needs to increase 150 mm when a 225 mm drainage structure is introduced.

In order to maintain the integrity and consistent benefit of the drainage structure, geosynthetics should be used as separators and filters around the drainage structure.

8.3 Design Procedure

In order to provide a defendable urban roadway design, *in situ* field state conditions, critical state traffic loading and scientific post construction validation need to be included in the design procedure. Modulus resilient evaluation of the aggregates is the standard currently being implemented in many agencies. Resilient modulus testing will allow for a more mechanistic based evaluation of the roadways. However, the next step required in urban design should include an evaluation based on varying loading frequencies as seen in urban settings. Using a combination of the resilient modulus testing that evaluates over different temperatures and dynamic modulus that evaluates over frequencies gives the most information to the designer. A catalogue of material characteristics should be developed matching the stress strain characteristics of the soil, aggregate and asphalt materials with the *in situ* field state conditions quantifiable with standard geotechnical tests. Once the material properties are catalogued within the model, standard geotechnical testing is all that is required to determine the optimal materials and thicknesses for each roadway design.

The designs can be optimized to provide the best structure in order to achieve the deflection requirements set by the City of Saskatoon asset management group. The roadways, once constructed, can then be tested with standard HWD testing to validate the design and confirm construction occurred as specified.

Including post construction testing is a key element in ensuring the designs are able to provide the service life required by the asset management team. The testing also provides validation information for designers when asked to provide documentation supporting their design decisions.

CHAPTER 9 SUMMARY AND CONCLUSIONS

9.1 Technical Feasibility Summary

The standard aggregate tests as outlined by the City of Saskatoon are not written with recycled aggregates in mind. While the recycled aggregates do pass some of the requirements of the standard specifications such as gradation, other tests were not designed for aggregates that have residual cohesion from asphalt or portland cement concrete. When the materials are evaluated for their mechanical characteristics the recycled materials performed better than the conventional aggregates.

According to conventional material characteristics, the recycled aggregates can pass City of Saskatoon specifications for gradation, aggregate angularity and plastic index. The sand equivalency test was not applicable to recycled aggregates due to the presence of cement fines.

The recycled aggregates did not meet the specification requirements for the standard CBR test. However, when the CBR test was modified so that the recycled aggregates were compacted with the gyratory compactor instead of the proctor hammer the recycled aggregates met the specification requirements. As the gyratory compactor is a more accurate representation for field aggregate compaction, these test results indicate an adequate granular material.

The soaked CBR test also evaluated the effect of the moisture on the aggregates through a percent swell component. Each of the gradations of recycled aggregates had a lower percent swell than conventional base aggregate.

When the aggregates were tested for their mechanistic characteristics the conventional base aggregate failed and a high fracture content base aggregate was also tested. The recycled aggregates outperformed the conventional aggregates by 80 to 200 percent. The recycled asphalt dynamic modulus was at least 200 percent higher than the high fracture conventional base aggregate. As the dynamic modulus is an indicator of the stiffness of the aggregate, the recycled aggregate layer could then be constructed thinner to provide the same structural capacity as a conventional structure.

Recycled asphalt cement aggregates as a base layer, while having a risk of higher permanent deformation, are stiffer than conventional aggregates and perform better at reducing strains in the subgrade when the subgrade is wet, even without a drainage layer. In the local structures the compressive strain on the top of the subgrade for the GB and GB (HF) structures increased by 125 percent and 146 percent, respectively, when compared to the RAP structure. Comparing the strains in the wet subgrade between the RAP and GB structures indicates that the RAP reduces the tensile strain on the bottom of AC by 88.6 percent and the compressive strain on the top of the subgrade by 78.3 percent.

The recycled portland cement concrete showed similar benefits when evaluated for its mechanistic characteristics. An increase in dynamic modulus of 80 to 100 percent was experienced in the recycled portland cement concrete OGBC aggregate when compared to the high fracture conventional base aggregate.

Creating an OGBC with the recycled asphalt and recycled portland cement concrete aggregates produced a granular material that had a significant reduction in moisture intake as well as lowering the electrical conductivity of the granular material. These attributes show a significant reduction in moisture sensitivity with the OGBC gradation when comparing to the standard base course gradation in the recycled aggregates.

When the RAP mechanistic materials were used in the FEM model in comparison to conventional base aggregate characteristics the RAP structure experienced lower strains in all structural layers. In the local structures with dry conditions, RAP reduced the shear strains between 25 and 51 percent through the structure. When the same structures were modeled on wet subgrades, the shear strains were reduced by 78 and 82 percent through the structure.

9.2 Stress Dissipation and Drainage Layer Summary

The drainage layer was modeled in the FEM and compared to the typical construction methodologies used by the City of Saskatoon. In local structures with wet subgrades, the strains were reduced by 66 to 92 percent throughout the structure when drainage is present. The drainage structure is 150mm thicker than the standard local structure. However, the strains in the drainage structure are still 64 to 89 percent lower than the granular structure of the same thickness.

In arterial structures on wet subgrades, the strains were reduced by 52 to 94 percent throughout the structure when drainage is present. The drainage structure is the same thickness as the standard arterial structure. When an additional 150mm of granular was added to the standard structure there was little to no effect on the strains identified by the model.

When the drainage structures were evaluated in field construction all roadways showed improved performance through non destructive testing. The deflection data showed that all segments ranged from fair to very poor according to the City of Saskatoon asset management system prior to reconstruction. Using drainage layers in the reconstruction brought all segments to either good or poor condition in the same rating system post construction.

The initial capital cost evaluation had the design local drainage structure 8 percent more costly than the thickened local structure currently used by the City of Saskatoon. The extra cost is due to the use of geosynthetics in the drainage layer. The arterial drainage structure is 4 percent less expensive than the thickened arterial structure.

The service life evaluation shows a significant reduction in the capital costs over an 80 year period. The local drainage structure had long term capital costs 34 percent less than the standard local structure and 30 percent less than thickened structure.

Evaluating the energy usage for the construction of the various structures showed that constructing a drainage structure for local roadways increased the energy usage by 23 percent using conventional aggregate for the base. Substituting RAP for the base aggregate had the construction using 5 percent less energy than a thickened local structure. The arterial drainage structure used 2 percent less energy to construct than the thickened arterial structure. When RAP was substituted for the conventional base in the drainage structure, the energy reduction was 13 percent compared to the thickened arterial structure.

9.3 Recommended Design Framework Summary

In order to include the best recycled aggregates in the City of Saskatoon specifications various changes and additions are required. Firstly, changes in the wording of the specifications allowing recycled aggregates as an alternative are required. As seen in Chapter 3, standard testing does accurately determine the structural adequacy of the recycled aggregate. Therefore, including a specification for a minimum dynamic modulus would allow for a good comparison between the various aggregate options.

Gyratory compaction methods should be implemented in evaluating the optimum compaction of aggregate layers. This would require a change in specification used by public agencies and testing agencies to acquire the equipment and training necessary to use the equipment. The top performing recycled aggregates, when evaluated by gradation, were classified as open graded base course. In order to be able to provide the best road structures the OGBC gradation should be included in the City of Saskatoon specifications.

Including a drainage layer in the design is critical to reduce the strains on the subgrade when moisture is present. A 225mm drainage layer constructed with crushed rock or crushed portland cement concrete allows for the drainage layer to take the place of a portion of the conventional aggregate typically used in the road structure.

Using drains within the structure presents significant maintenance and repair hurdles for agencies. In order to for the granular structure to perform the drains need to be maintained and kept free of debris. As well, the porosity of the drain needs to be high enough that moisture does not migrate up easily during frost action. As well, in urban evnironments the risk of underground utility breaks makes it essential that the repair crews are trained in drainage structure construction requirements and have the budget and materials in place to repair the drainage structure after the utility break. Testing in chapters 3 and 4 indicated that crushed portland cement concrete has all the qualities required to be a stress dissipating drainage layer if it is carefully processed. The test sections constructed with crushed portland cement concrete as a drainage layer all performed fair to good in the City of Saskatoon asset management testing framework.

To include recycled aggregates and drainage layers in the design protocols, the use of high caliber modelling software like the PSIPAVE3D™ to determine the optimal structure are required. The software used should have the capability of designing structures to meet deflection requirements of the COS asset management system that are easily validated through post construction HWD testing.

9.4 Future Research Summary

As identified in Chapter 3, the CBR testing for recycled aggregates is not adequately characterized using a proctor hammer to compact the samples. As the proctor hammer was used to simulate the use of a sheepsfoot roller, it is a better tool for subgrades. During construction, all aggregate layers are compacted with smooth rollers, which are more clearly simulated with gyratory compaction. An evaluation should be done to determine which lab compaction method is more appropriate for the various materials used in roadway construction.

With the well graded crushed content having a high fines content, soil water characteristic curves should be created to determine how the type of fines present in the aggregate react in the presence of water.

An evaluation of the long term effects of RAP under loading should be completed. The test sections with the base layer consisting of 100 percent RAP are currently four years old and should be evaluated periodically to determine if there is an increased risk of rutting in this type of construction. Some studies have shown that compaction occurs during loading, causing a risk of rutting in the granular layer.

While the recycled aggregates, when modeled, indicate an increased resistance to strain and superior structural characteristics, the relationship between resistance to strain and extension of service life is not known. Therefore, further research is required to quantify the exact life cycle benefits to using recycled aggregates within the structure.

As the post construction non-destructive deflection values were higher than expected due to high subgrade moisture values, the test sections should be evaluated over a longer period of time to determine if the deflection values decrease as subgrade moisture is removed by the drainage layer.

Other non destructive testing, such as ground penetrating radar can also be used to track the moisture migration over time and therefore evaluate the structural capacity of the roadway through known moisture – stiffness relationships.

LIST OF REFERENCES

ASTM C117, 2. (2004). Standard Test Method for Materials Finer than 75-μm (No. 200) Sieve in Mineral Aggregates by Washing. West Conshohocken, PA: ASTM International.

ASTM C136, 2. (2006). Standard Test Method for Sieve Analysis of Fine and Coarse Aggregates. West Conshohocken, PA: ASTM International.

ASTM D1883, 0. (2007). Standard Test Method for CBR (California Bearing Ratio) of Laboratory-Compacted Soils. West Conshohocken, PA: ASTM International.

ASTM D2419, 2. (2009). Standard Test Method for Sand Equivalent Value of Soils and Fine Aggregate. West Conshohocken, PA: ASTM International.

ASTM D2487, 2. (2011). Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System). West Conshohocken, PA: ASTM International.

ASTM D3282, 2. (2009). Standard Practice for Classification of Soils and Soil-Aggregate Mixtures for Highway Construction Purposes. West Conshohocken, PA: ASTM International.

ASTM D4318, 2. (2010). Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. West Conshohocken, PA: ASTM International.

ASTM D5821, 2. (2006). Standard Test Method for Determining the Percentage of Fractured Particles in Coarse Aggregate. West Conshohocken, PA: ASTM International.

Berthelot, C. (1999). *Mechanistic Modelling of Saskatchewan SPS-9A Asphalt Concrete Pavements.* Department of Civil Engineering, University of Saskatchewan.

Berthelot, C., Haichert, R., Podborochynski, D., Wandzura, C., & Taylor, B. (2011). An Integrated Mechanistic Based Framework for Sustainable "Green Street" Urban Low Volume Road Rehabilitation. *Transportation Research Record: Journal of the Transportation Research Board of the National Academies, 3*, 103-110.

Berthelot, C., Haichert, R., Podborochynski, D., Wandzura, C., Taylor, B., & Guenther, D. (2010). Mechanistic Laboratory Evaluation and Field Construction of Recycled Concrete Materials for Use in Road Substructures. *Transportation Research Record: Journal of the Transportation Research Board of the National Academies*, 41-52.

Berthelot, C., Haichert, R., Podborochynski, D., Wandzura, C., Taylor, B., & Guenther, D. (2011). Mechanistic Design and Non-Destructive Structural Validation of a

"Green Street" Test Sections Using Recycled Rubble Materials. *Transportation Research Record: Journal of the Transporation Research Board of the National Academies*, 111-117.

Berthelot, C., Haichert, R., Soares, R., Podborochynski, D., & Guenther, D. (2011). City of Saskatoon Mechanistic Pavement Structure Modeling. *Transportation Association of Canada Annual Conference.* Edmonton, AB.

Berthelot, C., Podborochynski, D., Kelln, R., Prang, C., Guenther, D., & Cherry, D. (2011). Comparison of Field Performance of Conventional and Recycled Drainage Systems using Non Destructive Structural Asset Management. *Journal of the Transportation Research Board of the National Academies*.

Berthelot, C., Taylor, B., Stuber, E., Prang, C., Marjerison, B., & Campbell, M. (2009). Use of Structural Asset Management to Evaluate Road Substructure Drainage Systems. *Transportation Research Record: Journal of the Transportation Research Board of the National Academies*, 44-52.

Canadian Strategic Highway Research Program. (2002). *Pavement Structural Design Practices Across Canada.* Ottawa: Canadian Strategic Highway Research Program.

City of Edmonton. (2009). City of Edmonton Specifications. Edmonton, Alberta, Canada.

City of Edmonton. (2010). *For Residents - Aggregate Recycling*. Retrieved May 23, 2010, from City of Edmonton Web Site: http://www.edmonton.ca/for_residents/service-aggregate-recycling.aspx

City of Saskatoon. (2008). *City of Saskatoon New Neighbourhood Design and Development Standards Manual.* Saskatoon.

City of Saskatoon. (2009). *Green Street Evaluation RFP.* RFP, Saskatoon.

City of Saskatoon. (2011). *2011 Standard Construction Specifications.* Saskatoon.

City of Winnipeg. (2010). *City of Winnipeg Specifications.* Winnipeg.

Duce, T. (2010). Correspondance.

Federal Highway Administration. (2004). *Recycled Concrete Aggregate.*

Foth, M., Guenther, D., Haichert, R., & Berthelot, C. (2010). City of Saskatoon's Green Streets Program - A Case Study for the Implemenation of Sustainable Roadway Rehabilitation with the Reuse of Concrete and Asphalt Rubble Materials. *ASCE Green Streets and Highways Conference.* Denver.

Gonzalez, G., & Moo-Young, K. (2004). *Transportation Applications of Recycled Concrete Aggregates.* FHWA.

Government of Ontario. (2008, December 9). *Ontario Provincial Standards for Roads and Public Works*. Retrieved 01 13, 2012, from http://www.raqsb.mto.gov.on.ca/techpubs/ops.nsf/d37f5a16d8174ffa85256d130066857f/fd614e3ba8c114858525706e00676aa5?OpenDocument

Guenther, D., Haichert, R., Foth, M., & Berthelot, C. (2011). Effect of Fines on the Structural Capacity of Base Course Materials. *Journal of the Transportation Research Board of the National Academies*.

Guthrie, W. S., Cooley, D., & Eggett, D. L. (2007). Effects of Reclaimed Asphalt Pavement on Mechanical Properties of Base Material. *Transportation Research Record: Journal of the Transportation Research Board*, 44-52.

Haichert, R., Foth, M., Guenther, D., & Berthelot, C. (2011). Case Study Evaluation of the Sustainability of Recycled Aggregate Road Structure. *Transportation Research Board.* Transportation Research Record.

Haichert, R., J., C., Bajpai, A., & Berthelot, C. (2009). Eco-Street: Quantifying Energy Efficiency of Roads over their Lifespan. *Transportation Association of Canada Annual Conference* (pp. 18-21). Vancouver: CDROM Proceedings.

Holtz, R. D., & Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering.* Englewood Cliffs: Prentice-Hall.

Kim, W., & Labuz, J. F. (2007). *Resilient Modulus And Strength Of Base Course With Recycled Bituminous Material.* St Paul: Minnesota Department of Transportation.

Lacher, C. (2006). *Moisture Movement in Highway Pavement Structures Coupled With Soil-Atmospheric Fluxes.* Saskatoon: University of Saskatchewan.

Li, Q., Xiao, D. X., Wang, K. C., Hall, K. D., & Qiu, Y. (2011). Mechanistic-empirical pavement design guide (MEPDG): a bird's-eye view. *Journal of Modern Transportation*, 114-113.

NCHRP. (1997). *NHCRP Synthesis 239: Pavement Subsurface Drainage Systems.* National Cooperative Highway Research Program.

Ostrander, J. (2009, September 12). Roadway Inspector. (D. Guenther, Interviewer)

Pavement Interactive. (2007, August 13). *AASHO Road Test*. Retrieved July 26, 2013, from Pavement Interactive: http://www.pavementinteractive.org/article/aasho-road-test/

PIARC Technical Committee C4.3 Road Pavements. (2008). *Review of the Growth and Development of Recyling in Pavement Construction.* World Road Association.

Prang, C. (2012). *Integration of Structural Information into an Urban Asset Management System.* Saskatoon: College of Engineering, University of Saskatchewan.

PSI Technologies Inc. (2010). *Sustainable "Green Street" Road Rehabilitation Using Recycled Asphaltic Concrete, Portland Cement Concrete, and Other Waste Rubble Materials, Year One Final Report.* Saskatoon.

Recycled Materials Resource Center. (2008, July 28). *Reclaimed Asphalt Pavement User Guideline*. Retrieved May 25, 2010, from Recycled Materials Resource Center : http://www.rmrc.unh.edu/tools/uguidelines/rap132.asp

Saskatchewan Ministry of Highways and Infrastructure. (2009). *Surfacing Manual.*

Sayed, S. M., Pulsifer, J. M., & Jackson, N. M. (2011). UNRAP: Are We Ready for It? *Journal of Materials in Civil Engineering*, 188-196.

Schimmoller, V. E., Holtz, K., Eighmy, D. T., Wiles, C., Smith, M., Malasheskie, G., et al. (2000). *Recycled Materials in European Highway Environments: Uses,.*

Scullion, T., & Saarenketo, T. (1997). Using Suction and Dielectric Measurements as Performance Indicators for Aggregate Base Materials. *Transportation Research Record: Journal of the Transportation Research Board*, 37-44.

Shosha, M. R. (n.d.). *Classification of Soil*. Retrieved April 14, 2012, from Soil Properties: http://osp.mans.edu.eg/geotechnical/Ch1%20A.htm

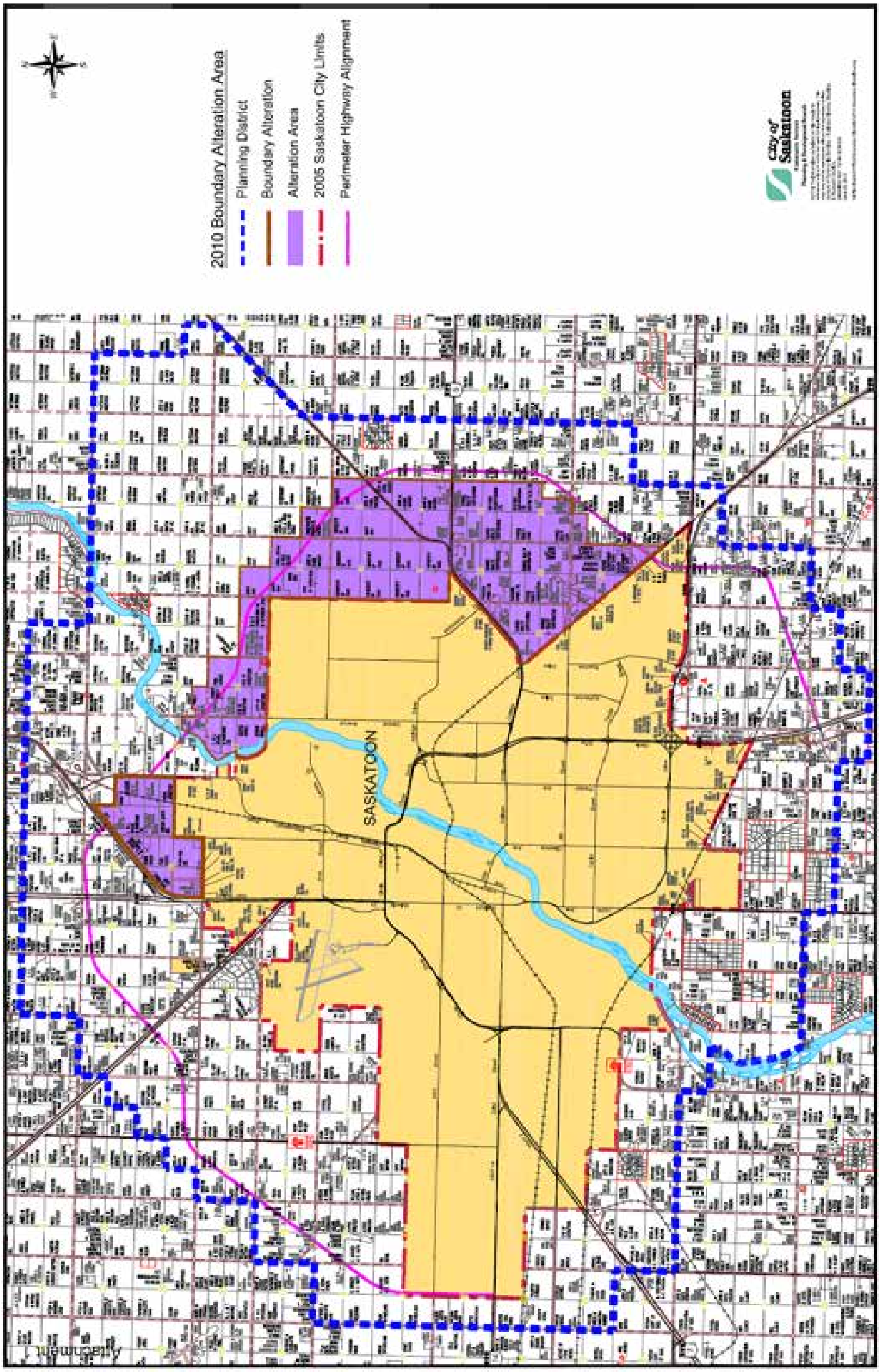
Texas Department of Transportation. (2008, September). *Recycled Concrete Aggregates Make Cents*. Retrieved July 26, 2013, from Texas Department of Transportation Recycled Concrete Aggregate: http://ftp.dot.state.tx.us/pub/txdot-info/gsd/concrete_0-1753.pdf

Thomas, L. (2009). *Mechanistic-Emperical Equivalent Single Axle Loads for Urban Pavements.* Saskatoon: University of Saskatchewan.

Transporation Association of Canada. (1997). *Pavement Design and Management Guide.* Ottawa: Transportation Association of Canada.

Transporation Research Board Pavement Management Section. (2007). *Pavement Lessons Learned from the AASHO Road Test and Performance of the Interstate Highway System.* Washington: Transportation Research Board.

APPENDIX A
CITY OF SASKATOON DEVELOPMENT MAP


2010 Boundary Alteration Area
Planning District
Boundary Alteration
Alteration Area
2005 Saskatoon City Limits
Perimeter Highway Alignment
SASKATOON
City of Saskatoon
Attachment 1

APPENDIX B
CITY OF SASKATOON AGGREGATE SPECIFICATIONS

INDEX

03001-1 DESCRIPTION

Work under these specifications shall include the supplying, processing, stockpiling, loading, hauling and dumping or spreading of granular material meeting the requirements shown for each classification. Delivery shall be to anywhere within the City of Saskatoon.

03001-2 REFERENCE TO STANDARD SPECIFICATIONS

Reference in these Specifications will be made to the latest edition of the American Society for Testing Materials (A.S.T.M.) and Canadian Standards Association (C.S.A.) covering aggregate and methods of testing aggregates.

03001-3 MATERIALS

3.1 General

3.1.1 Source

The source of the aggregate shall be the locations specified in Schedule "E" - List of Production Locations in the Tender Form or alternate locations acceptable to the Engineer.

3.1.2 Composition

The aggregate shall consist of fragments of durable rock, free from undesirable quantities of soft or flaky particles, shale, loam, vegetation or other deleterious material.

3.1.3 Moisture Content

Except when moisture control is specified, aggregate moisture content (by dry aggregate weight) shall be in the following ranges:

- Maintenance Base Aggregate, 3-6%;
- Dry Maintenance Base Aggregate, 0-3%;
- Street Sanding aggregate, 0 - 4%
- for all other aggregates, 0 - 5%

When moisture control is specified it shall be carried out as a part of processing and stockpiling, and shall be included in the unit costs. Such aggregate (moisture control specified) shall, at time of delivery to site, have a moisture content in the range of 1.5% below optimum to 0.5% above optimum.

All aggregate delivered to the job site, having a moisture content greater than specified in 3.1.3 will be subject to a price adjustment from the contract unit price. The price reduction will be $0.50 per percent moisture for street sand, and $0.40 per tonne per percent moisture for all other aggregate over the maximum specified.

Example 1 – Street Sand

For 2,000 tonnes street sanding aggregate with a moisture content of 6% would be:

Actual	Difference	Rate Reduction
6.0%	6.0% - 4.0% = 2.0%	2,000t x $0.50/t/% x 2% = $2,000.00

Example 2 – Moisture Control

City orders 2,000 tonnes of base aggregate to be supplied at optimum moisture content (use 7.2%), the corresponding payment reduction for material delivered with a moisture content of 8.0% and 4.9% would be:

Actual	Allowable Difference
8.0%	7.2% + 0.5% = 7.7%
	Rate Reduction 2,000t x (8.0%-7.7%)*$0.40/t/% = $240.00
4.9%	7.2% - 1.5% = 5.7%
	Rate Reduction 2,000t x (5.7%-4.9%)*$0.40/t/% = $640.00

3.1.4 Existing Stockpiles

Material stockpiled by the Contractor both prior to and after the award of the Contract will not be accepted unless:

1. testing was carried out by an approved Testing Agency at the minimum frequency specified in Section 3.3.1.
2. test results indicate the material meets current specifications and is uniform throughout the stockpile, and;
3. random testing by the City at time of delivery confirms the quality of the material.

3.2 Classification

3.2.1 Subbase Aggregate

Subbase aggregate shall be uniformly graded between the following limits:

Sieve Designation	Percent by Weight Passing
50 mm	100
25 mm	75-100
12.5 mm	52-100
5 mm	30-75
2 mm	20-55
400 µm	8-30
71 µm	3-15

The Plasticity Index of the material passing the 400 um sieve shall not exceed 6.

The organic content of the material passing the 5 mm sieve shall not exceed 3.0% by weight.

The material, when compacted to 100% of the maximum density as determined by the Standard Proctor Compaction Test, shall have a minimum CBR of 25 in the unsoaked condition at 0.1" or 0.2" penetration, whichever is greater (ASTM D1883).

3.2.2 Base Aggregate

Base aggregate shall consist of a homogenous mixture of crushed gravel, sand filler and clay binder with a maximum organic content of 1.0% by weight, and shall be uniformly graded between the following limits:

Sieve Designation	Percent by Weight Passing
25 mm	100
18 mm	87-100
12.5 mm	72-93
5 mm	45-77
2 mm	29-56
900 µm	18-39
400 µm	13-26
160 µm	7-16
71 µm	6-11

At least 50% by weight of the material retained on the 5 mm sieve shall have one or more fractured faces created by the crushing operation. The organic content of the material passing the 5 mm sieve shall not exceed 3.0% by weight.

The material, when compacted to 100% of the maximum density as determined by the Standard Proctor Compaction Test, shall have a minimum CBR of 65 in the unsoaked condition at 0.1" or 0.2" penetration whichever is greater (ASTM D1883).

3.2.3 Asphalt Aggregate

Specifications for all types of asphalt aggregate are contained in the Asphalt Specifications Section 04010 "Asphalt Mix".

3.2.4 Street Sanding Aggregate

Street sanding aggregate shall consist of clean, hard, durable particles free from clay, loam and other objectionable material.

The aggregate shall contain a minimum 25% crushed and/or angular aggregate particles (by weight) retained on the 2.5mm plus sieve.

The aggregate shall be free from frozen lumps under all weather conditions. It will be the Contractor's responsibility to protect his stockpiles from excessive moisture, to waste frozen material, or to take whatever steps necessary to meet this requirement.

Street sanding aggregate shall meet the following grading limits:

Sieve Designation	Percent by Weight Passing
9 mm	100
5 mm	87-95
2.5 mm	50-70
900 um	25-40
400 um	7-22
160 um	0-5
71 um	0-3

The minimum street sanding aggregate requirements are to be met by October 1st.

Sieve analysis and moisture content testing will be performed on samples obtained from the conveyor prior to processing with salt or liquid de-icer. The following payment adjustments for street sand will apply as follows:

a) More than 3% passing by weight on the 71μm sieve

% Passing by Weight	Payment
0% to ≤ 3.2%	100%
> 3.2% to ≤ 3.9%	90%
> 3.9% to ≤ 5.0%	80%
> 5.0% to ≤ 6.0%	70%
> 6.0%	0%

b) Less than 100% passing by weight on the 9mm sieve (these adjustments can be applied to samples taken at point of deliver with or without de-icer additives.

% Passing by Weight	Payment
≥ 99.5%	100%
> 99.0% to < 99.5%	90%

> 98.0% to ≤ 99.0%	80%
≤ 98.0%	0%

c) Material meeting specifications for percent passing the 9mm and 71 µm sieve, but out of specification on any other sieve size. A payment reduction of $0.10/tonne will be assessed against all of the material processed that day.

3.2.5 Concrete Aggregate

Specifications for concrete aggregate are contained in Concrete Specifications Section 06005 "Ready Mixed Concrete".

3.2.6 Plaster Sand

The aggregate shall consist of fine granular material composed of hard, strong, durable mineral particles which are free of injurious amounts of saline, alkaline, organic or other deleterious substances.

The grading shall be from fine to coarse within the following limits:

Sieve Designation	Percent by Weight Passing
5 mm	100
2.5 mm	95-100
1.25 mm	85-96
630 µm	68-93
280 µm	27-79
160 µm	0-59
71 µm	0-9

3.2.7 Pipe Bedding Aggregate

Pipe bedding aggregate shall conform to the following gradation:

Sieve Designation	Percent by Weight Passing
19.00 mm	100
12.50 mm	25-100
4.75 mm	45-70
2.00 mm	28-50
850 µm	18- 36
425 µm	12- 26
150 µm	7- 15
75 µm	5- 10

3.2.8 Crushed Rock

Crushed rock shall be composed of fragments of durable rock, free from undesirable quantities of soft or flaky particles, shale, loam and other deleterious material.

The material shall conform to the following grading limits:

Sieve Designation	Percent by Weight Passing
50 mm	100
25 mm	0-80
12.5 mm	0-18
5 mm	0-12
71 µm	0-5

At least 50% by weight of the material retained on the 5 mm sieve shall have one or more fractured faces created by the crushing operation. Crushed rock shall be delivered as required to one of the City of Saskatoon material reclamation yards, or, if so directed by the Engineer, to the City Yards, or other alternate location.

3.2.9 Pit Run

Shall be of durable aggregate free from deleterious material such as roots, grasses, and topsoil and have a top size of no greater than 150 mm and have a minimum of 35% by weight retained on the plus 5 mm sieve.

3.2.10 Non-Shrink / Unshrinkable Fill

Non-shrink/unshrinkable fill shall conform to the following specifications:

- 28 day Compressive Strength - 0.30 to 0.50 MPa.
- Strength after 24 hours - a minimum of 0.07 MPa.
- Binder - Type 10 Portland Cement - minimum 30 Kg per cubic metre.
- Air entrainment - 5% to 8%.
- Slump - 150 mm to 200 mm.
- Aggregate shall be a type used for concrete, consisting of clean, hard durable stone or gravel free from lumps, soft and flaky particles, organic matter, salt, alkali and adherent coatings. No more than <u>10%</u> by weight of the aggregate shall be finer than passing the 75 µm sieve.

3.2.11 Bedding Sand

The aggregate shall consist of fine granular material composed of hard, strong, durable mineral particles which are free of injurious amounts of saline, alkaline, organic or other deleterious substances.

The grading shall be from fine to coarse within the following limits:

Sieve Designation	Percent by Weight Passing
5 mm	95-100
2.5 mm	80-100
1.25 mm	50-85
630 µm	25-60
280 µm	10-30
160 µm	0-15
71 µm	0-5

3.3 Testing

3.3.1 Tests and Frequency

All tests shall be carried out in accordance with current ASTM or CSA Standards. Test frequencies shall apply both to production (quality control) and to delivery (quality assurance).

Material	Minimum Frequency of Test	Test Required
Base and subbase	Every 2,000 tonnes	1. Wash Sieve Analysis 2. Percentage Crush 3. Moisture Content
	Every 6,000 tonnes	1. Standard Proctor 2. CBR Value 3. Plasticity Index 4. Organic Content
Sanding Aggregate	Every 1,000 tonnes	1. Wash Sieve Analysis 2. Moisture Content
Plaster Sand	Every 200 tonnes	1. Wash Sieve Analysis
Pipe Bedding Sand	Every 2,000 tonnes Every 6,000 tonnes	1. Wash Sieve Analysis 1. Organic Content
Crushed Rock	Every 2,000 tonnes	1. Wash Sieve Analysis

3.3.2 Testing Services

Sampling and laboratory testing will be conducted by an agency appointed by the City. Copies of all test results will be made available to the Contractor.

In addition, the Contractor may choose to employ his own testing agency at his cost. However, in the event of a discrepancy between test results, those obtained by the City's testing agency will govern.

3.3.3 Notification

The Contractor shall notify the Engineer at least one working day prior to the commencement or the resumption of aggregate production. Whenever possible, sampling will be done from the crusher belt.

3.3.4 Costs

The Contractor shall bear the cost of sampling and testing material in the following situations:

1. Initial testing to bring production into specification requirements.
2. Testing as above when source of material is changed.
3. Retesting of material which failed to meet specifications.

Testing for 1 and 2 may be conducted by the Contractor's agency. If the City conducts the testing, the costs will be deducted from any subsequent progress payments.

The cost of other testing initiated by the City during production will be borne by the City. No compensation will be made to the Contractor for testing initiated by the Contractor during current or previous production.

3.3.5 Sieve Analysis

The gradation of the material, when plotted on a semi-log grading chart, shall appear as a smooth curve within the specified band.

The average of the results of any 5 consecutive Wash Sieve Analysis Tests on material sampled at the crusher belt, or any 2 consecutive Wash Sieve Analysis Tests on material sampled at the delivery location, shall be within the grading limits specified for that material. Failure to meet this requirement shall result in the rejection of the material.

3.4 Enforcement of Specifications

Delivery of material to City locations will not be permitted until test results confirm that it meets specifications, and until the stockpile requirements of Section 5.2 have been fully met.

Any deviation from specifications during the production of material shall require the Contractor to take immediate corrective action. Equipment shall be shifted to ensure that there is no contamination of the current stockpile. A new stockpile shall be started adjacent to the former stockpile only after the product has been proven by testing to again meet specifications. Any material of inferior quality, or not in accordance with this specification, brought to, or incorporated into the work shall be immediately removed by the Contractor, at his own expense. In the event of the Contractor failing to comply with this provision, the Engineer may remove such materials, or cause them to be removed and deduct the cost of same from any subsequent progress payments to the Contractor.

In the event that removal of the inferior material is not required by the Engineer for any reason, then an appropriate payment adjustment as defined by the Engineer shall be applied to all of that material delivered to the site on that day.

03001-4 EQUIPMENT

4.1 Weigh Scales

The City will provide weigh scales at no cost to the Contractor at the City Yards at Ontario Avenue and 26th Street on a year round basis.

The Contractor may weigh large and/or continuous deliveries at the scale most convenient to his source. Small or intermittent deliveries outside the normal construction season shall be weighed over Scale at the City Yards.

The Contractor shall, where indicated in the tendering documents, quote on using his own scale, provided that the scale is certified.

4.2 Trucks

The Contractor may use any trucks of any type capable of delivering in accordance with job requirements. Specific conditions to be met include:

1. Certain delivery points are confined areas such as lanes and parking lots which may prevent the use of trucks larger than tandems.
2. Delivery is either spreading for road construction or dumping in a manner suitable for the project.
3. Centre-dump vehicles will only be permitted where base or subbase is to be spread for roadway construction.

Prior to the start of delivery, the Contractor shall supply a complete list of trucks, owner's names, registration numbers, tare weights and licence load limits. This list shall be updated whenever changes occur.

All trucks shall be weighed when delivery commences and at random times during the Contract.

03001-5 CONSTRUCTION

5.1 Production

5.1.1 Blending

Care shall be taken in the selection of material in the pit so as to produce a uniform product.

If blending of materials from more than one source is required to meet specifications, all such blending shall be done in the production equipment.

5.1.2 Sand Elimination

When it is necessary to eliminate sand to meet the grading specifications, the sand shall be removed prior to the crushing operation.

5.2 Stockpiling

5.2.1 Procedure

Each stockpile shall be constructed to contain not less than 10,000 tonnes or one-half of the remaining estimated contract quantity (whichever is less).

The area where the stockpile is to be located shall be shaped to a uniform smooth surface and graded to ensure positive drainage from the stockpile.

The material shall be placed uniformly on a predetermined area, in layers not exceeding 1 m in thickness.

Construction operations shall be controlled to prevent segregation of the various particle sizes.

If material is dumped by vehicles, it shall be spread with a dozer. The construction of each layer shall progress from outer edges toward the centre.

The material shall not be pushed or dumped over the edges or down the faces of the stockpile.

The material may be stockpiled from a stacker or conveyor belt only if all material is transported from the conveyor belt across the stockpile by means of a dozer or loader.

At the end of each day, the top of the stockpile and gravel pit shall be properly levelled and sloped. When stockpiling is carried out in winter, the Contractor shall take precautions that no snow is incorporated into the stockpile.

The completed stockpile shall be neat and regular in form and shall be constructed to occupy the smallest feasible area.

If different types of material are to be stockpiled, the piles shall be located and constructed so that no intermingling of material will occur.

Any rejected material must be placed a good distance away from an approved stockpile.

Material which does not meet specification within 5,000 tonnes of crushing will therefore be rejected and a new stockpile will be started in a new location clearly away from the rejected material.

5.2.2 Minimum Quantities

During the construction season, the Contractor shall, on his site, maintain the following minimum stockpile quantities of stockpiled, tested and approved material:

Base Aggregate	15,000 tonnes
Subbase Aggregate	15,000 tonnes
Sanding Aggregate	10,000 tonnes
Other Material	5,000 tonnes

All material supplied shall be loaded from a stockpile. Direct delivery from the crusher will not be permitted.

5.3 Delivery

5.3.1 Rate of Delivery

The Contractor shall be prepared to supply the following approximate quantities at a uniform delivery rate during each working day (including Saturdays when prior notice is given).

1. Granular Base and Subbase

If one Contractor has the contract to supply aggregate to both sides of the river, the Contractor shall be prepared to deliver up to 2,000 tonnes of aggregate per day to each side of the river or a total up to 4,000 tonnes per day. If a Contractor has only to supply aggregate to one side of the river, the Contractor shall be prepared to deliver up to 3,000 tonnes of aggregate per day.

2. Other Materials

The Contractor shall be prepared to deliver up to 1,000 tonnes per day of any other aggregate materials.

The Contractor and the Engineer shall be in close contact with the progress of the projects to determine daily delivery requirements. Whenever an appreciable change in delivery quantities or actual requirements is known in advance by either party, one party shall give reasonable notice to the other party.

5.3.2 Breakdown

Whenever the Contractor is unable to deliver any material due to major stationary plant or equipment (trucks and self-mobile equipment excluded) breakdown, the Contractor shall immediately notify the Engineer when delivery of material shall resume. Non-delivery time period for material required by the City shall not be longer than 24 hours.

5.3.3 Alternate Sources

In the event that the Contractor fails to supply any or only supplies a portion of material required, the Engineer under this Contract, shall be at liberty to purchase the required material so in default from any firm which is willing and ready to supply. The Contractor shall pay the City, on demand, any increase in the cost of material so purchased, over and above the cost of similar material under this Contract.

5.3.4 Loading Procedure

The Contractor shall provide supervision of the loading operation to ensure correct source and procedures. Material loaded from the stockpile shall be removed in a manner which results in mixing of the full height of the stockpile face.

5.3.5 Load Limits

Maximum gross weight of vehicles operating within the City limits shall conform to Section 7 of the Traffic Bylaw No. 7200 and to any current amendments. In addition, the load limit as established by the Saskatchewan licence for each vehicle shall not be exceeded.

Where rural municipal roads are to be used, requirements established by the R.M. Council must be met. This includes road maintenance and load permits.

5.3.6 Truck Routes

Vehicles operating under this Contract shall be confined to routes shown on Schedule No. 8, Division 0, Section 00705 of General Conditions. For purposes of making a delivery, trucks must stay on a designated route to the point closest to the delivery point. The Engineer has the right to assign the final portion of the route, namely over local streets from the designated route to the delivery point.

5.3.7 Maintenance of Haul Routes

The City will, at its own expense, maintain all haul routes within the City limits.

Haul routes outside the limits of the City of Saskatoon shall, insofar as practical with respect to minimizing haul distance, be on numbered provincial Highways.

Where hauling is required over roads outside the City other than along Provincial Highways, the Contractor shall, prior to commencing this Contract, make formal arrangements for the use of such roads with the municipality having jurisdiction. These shall include the extent of the Contractor's responsibility for maintenance of road surface, for traffic safety and for dust palliation. A copy of the agreement shall be submitted to the City prior to commencement of the Contract.

03001-5 MEASUREMENT

6.1 Aggregate

The unit of measurement shall be the tonne. If City or Contractor's weigh scales are not in operation, the aggregate weight shall be calculated on the basis of actual volume of material delivered and average unit weight determined from previously weighed truck loads of similar material.

6.2 Moisture

The moisture content (dry weight basis) shall be determined by sampling the material at the place of delivery and averaging the results of each one-month period. This average moisture content shall be applied against the total quantity delivered for that same one-month period to calculate a payment reduction.

6.3 Quantity Summaries

The Contractor shall submit monthly statements for each item showing a daily sub-total and a cumulative total for the period.

END OF SPECIFICATION 03001

APPENDIX C
TEST SECTION CONSTRUCTION PHOTOS

Figure C.1 Marquis Drive Rotomixing

Figure C.2 Rotomixer Teeth

Figure C-3 Marquis Drive Drainage Layer

Figure C-4 Marquis Drive Geosynthetics and RAP

Figure C-5 Marquis Drive Paving

Figure C-6 115th Street Rotomixing

Figure C-7 115th Street Geosynthetics and Drainage Aggregate

Figure C-8 115th Street Paving

Figure C-9 Kenderdine #1 Excavation and Geosynthetics

Figure C-10 Kenderdine Drainage Aggregate

Figure C-11 Kenderdine Emulsion Blending

Figure C-12 8th Street Rotomixing

Figure C-13 8th Street Geosynthetics

Figure C-14 8th Street Drainage Aggregate

Figure C-15 8th Street RAP layer

Figure C-16 8th Street Paving

Figure C-17 Field House Road Excavation and Geosynthetics

Figure C-18 Field House Road RAP Layer

Figure C-19 Field House Road Final Grading

Figure C-20 Field House Road Paving

Figure C-21 Kenderdine #2 Excavation

Figure C-22 Kenderdine #2 RAP layer

Figure C-23 Kenderdine #2 Final Grading

Figure C-24 Kenderdine #2 Paving

Figure C-25 Wilkinson Excavation

Figure C-26 Wilkinson Geosynthetics

Figure C-27 Wilkinson Aggregate layers

Figure C-28 Wilkinson Final Grading and Emulsion Blending

Figure C-29 Wilkinson Paving

Figure C-30 Adolph Drainage layer

Figure C-31 Adolph Geosynthetic and RAP Surface

Figure C-32 Adolph Final Grading

Figure C-33 Adolph Paving

APPENDIX D
CROSS SECTION CAPITAL COST BREAKDOWN

Table D-1 Local Roadway Layer Thicknesses

	Layer Thicknesses (mm)				
	Baseline	Baseline (RAP)	Thickened	Drainage Structure	Drainage Structure (RAP)
HMAC	45	45	45	45	45
Conventional Base	225	0	375	150	0
RAP	0	225	0	0	150
Sub-base	0	0	0	0	0
Crushed Concrete	0	0	0	225	225
Subgrade Prep	150	150	0	0	0

Table D-2 Local Roadway Unit Costs

Item	Unit	Unit Cost
HMAC	tonne	$ 160.00
Conventional Base	tonne	$ 45.00
RAP	tonne	$ 30.00
Sub-base	tonne	$ 42.00
Crushed Concrete	tonne	$ 40.00
Subgrade Prep	sq.m.	$ 10.00
Geosynthetics	sq.m.	$ 4.00
Drainage Pipe	sq.m. **	$ 6.60

** based on pipe along each side of a 10m roadway

Table D-3 Local Roadway Structure Cost Breakdown

Item	Baseline	Baseline (RAP)	Thickened	Drainage Structure	Drainage Structure (RAP)
HMAC	$ 17.28	$ 17.28	$ 17.28	$ 17.28	$ 17.28
Conventional Base	$ 22.28	$ -	$ 37.13	$ 14.85	$ -
RAP	$ -	$ 14.85	$ -	$ -	$ 9.90
Sub-base	$ -	$ -	$ -	$ -	$ -
Crushed Concrete	$ -	$ -	$ -	$ 16.20	$ 16.20
Subgrade Prep	$ 10.00	$ 10.00	$ -	$ -	$ -
Geosynthetics	$ -	$ -	$ 4.00	$ 8.00	$ 8.00
Drainage Pipe	$ -	$ -	$ -	$ 6.60	$ 6.60
Total	**$ 49.56**	**$ 42.13**	**$ 58.41**	**$ 62.93**	**$ 57.98**

Table D-4 Arterial Roadway Layer Thicknesses

Item	Layer Thicknesses (mm) Baseline	Baseline (RAP)	Thickened	Drainage Structure	Drainage Structure (RAP)
HMAC	110	110	110	110	110
Conventional Base	150	0	300	200	50
RAP	0	150	0	0	150
Sub-base	300	300	300	0	0
Crushed Concrete	0	0	0	250	250
Subgrade Prep	300	300	0	0	0

Table D-5 Arterial Roadway Unit Costs

Item	Unit	Unit Cost
HMAC	tonne	$ 160.00
Conventional Base	tonne	$ 45.00
RAP	tonne	$ 30.00
Sub-base	tonne	$ 42.00
Crushed Concrete	tonne	$ 40.00
Subgrade Prep	sq.m.	$ 10.00
Geosynthetics	sq.m.	$ 4.00
Drainage Pipe	sq.m. **	$ 6.60

** based on pipe along each side of a 10m roadway

Table D-6 Arterial Roadway Structure Cost Breakdown

Item	Baseline	Baseline (RAP)	Thickened	Drainage Structure	Drainage Structure (RAP)
HMAC	$ 42.24	$ 42.24	$ 42.24	$ 42.24	$ 42.24
Conventional Base	$ 14.85	$ -	$ 29.70	$ 19.80	$ 4.95
RAP	$ -	$ 9.90	$ -	$ -	$ 9.90
Sub-base	$ 27.72	$ 27.72	$ 27.72	$ -	$ -
Crushed Concrete	$ -	$ -	$ -	$ 18.00	$ 18.00
Subgrade Prep	$ 10.00	$ 10.00	$ -	$ -	$ -
Geosynthetics	$ -	$ -	$ 4.00	$ 8.00	$ 8.00
Drainage Pipe	$ -	$ -	$ -	$ 6.60	$ 6.60
Total	**$ 94.81**	**$ 89.86**	**$ 103.66**	**$ 94.64**	**$ 89.69**

APPENDIX E
ENERGY USAGE EVALUATION

	Baseline	Baseline- RAP	Thickened	Drainage	Drainage - RAP
Structural Composition:					
Virgin HMAC Thickness	45 mm	45 mm	45 mm	45 mm	45 mm
Virgin Base Thickness	225 mm	--	375 mm	150 mm	
Hauled-in Recycled Asphaltic Concrete Thickness	--	225 mm	--	--	--
Hauled-in Recycled Portland Cement Concrete Thickness	--	--	--	225 mm	225 mm
In-Place Recycled Base	--	--			150 mm
Total Structure Thickness	**270 mm**	**270 mm**	**420 mm**	**420 mm**	**420 mm**
Structural 'Stiffness':					
Virgin HMAC	77	77	77	77	77
Virgin Base	154	0	257	103	0
Hauled-in Recycled Asphaltic Concrete	0	191	0	0	0
In-Place Recycled PCC	0	0	0	315	315
In-Place Recycled Base	0	0	0	0	54
'Total' Structure Stiffness	**231**	**268**	**334**	**495**	**447**
Haul Quantities:					
Virgin HMAC	450 m³	450 m³	450 m³	450 m³	450 m³
Virgin Base	2250 m³	--	3750 m³	1500 m³	--
Stockpile Crushed Recycled Asphaltic Concrete	--	2250 m³	--	--	--
Stockpile Crushed Portland Cement Concrete				2250 m³	2250 m³
Cement for Base Stabilization	--	--	--	--	--
Aggregate for HMAC	410 m³	410 m³	410 m³	410 m³	410 m³
Asphalt Oil for HMAC	23 m³	23 m³	23 m³	23 m³	23 m³
Waste Granular	2700 m³	2700 m³	2700 m³	2700 m³	450 m³
Waste Subgrade	--	--	1500 m³	1500 m³	1500 m³
On-site Construction Equipment Resources					
Haul of Waste Granular/HMAC	7364 km	7364 km	7364 km	7364 km	0 km
Haul of Waste Subgrade	0 km	0 km	-4091 km	4091 km	4091 km
Haul of Recycled Portland Cement Concrete	0 km	0 km	0 km	4091 km	4091 km
Haul of Recycled Asphalic Concrete	0 km	6136 km	0 km	0 km	0 km
Haul of Virgin Base	8182 km	0 km	13636 km	5455 km	0 km
Haul of Aggregate to HMAC Plant	1117 km	1117 km	1117 km	1117 km	1117 km
Haul of Asphalt Oil to HMAC Plant	2864 km	2864 km	2864 km	2864 km	2864 km
Haul of HMAC to Site	818 km	818 km	818 km	818 km	818 km
Total Truck Kilometers (Round Trip)	**20344 km**	**18299 km**	**21708 km**	**25799 km**	**12980 km**

	Baseline		Baseline - RAP		Thickened		Drainage		Drainage - RAP	
	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)
Existing Road Structure										
HMAC Thickness (mm)	45		45		45		45		45	
Base Thickness (mm)	225		225		225		225		225	
Length of section for rehabilitation (m)	1000		1000		1000		1000		1000	
Width of road (m)	10		10		10		10		10	
Demolition of Existing Structure										
Breaking up and removing of original structure	M315D Wheel Excavator	219	M315D Wheel Excavator	219	M315D Wheel Excavator	219	M315D Wheel Excavator	219	RM-500 Rotary Mixer	64
Waste Stucture									950H Wheel Loader	24
HMAC & Base removal qty (m3)	2700		2700		2700		2700		450	
Subgrade removal qty (m3)	0		0		1500		1500		1500	
haul to landfill	367 Peterbilt	2945	367 Peterbilt	2945	367 Peterbilt	2945	367 Peterbilt	2945	367 Peterbilt	491
Subgrade preparation										
percent cement added (%)	0		0		0		0		0	
Production of cement materials	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from cement source to site	N/A		N/A		N/A		N/A		N/A	
Mixing of subgrade cement	N/A		N/A		N/A		N/A		N/A	
Shaping and compaction (hrs)	N/A		N/A		N/A		N/A		N/A	
Base										
Virgin Granular materials										
Quantity (m3)	2250		0		3750		1500		0	
Aggregate production (removal from pit and crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from aggregate source to site	367 Peterbilt	3273	367 Peterbilt	0	367 Peterbilt	5455	367 Peterbilt	2182	N/A	
Stockpiled Recycled Granular Materials										
Quantity (m3)	0		2250		0		0		0	
Recycled materials production (crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from recycled materials stockpile to site	N/A		N/A		N/A		N/A		N/A	
In-Place Recycled Granular Materials										
Soil Quantity (m3)	0		0		0		0		1500	
Milling* (May be captured category in demolition above)	N/A		N/A		N/A		Accounted Above		Accounted Above	
Moving of recycled granular to remove subgrade from below	N/A		N/A		N/A		N/A		367 Peterbilt	2182
Drainage Layer RCP Materials										
Soil Quantity (m3)	0		0		0		2250		2250	
Recycled materials production (crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from recycled materials stockpile to site	N/A		N/A		N/A		367 Peterbilt	3273	367 Peterbilt	3273
Modifications to in-place Granular Materials										
Soil Quantity (m3)	0		0		0		0		1500	
Percent cement	N/A		N/A		N/A		0%		0%	
Cement Quantity (m3)	0		0		0		0		0	
Production of cement materials	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
Haul distance from cement source to site	N/A		N/A		N/A		N/A		N/A	0
Percent emulsion	N/A		N/A		N/A		N/A		N/A	
Production of emulsion material	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from emulsion source to site	N/A		N/A		N/A		N/A		N/A	
Mixing of base & cement and/or emulsion	N/A		N/A		N/A		RM-500 Rotary Mixer	0	RM-500 Rotary Mixer	169
Shaping and compaction	12M Motor Grader,	163	12M Motor Grader,	163	12M Motor Grader,	272	12M Motor Grader,	109	12M Motor Grader,	217
	CP76 Vibratory Soil Compactor,	66	CP76 Vibratory Soil Compactor,	66	CP76 Vibratory Soil Compactor,	110	CP76 Vibratory Soil Compactor,	44	CP76 Vibratory Soil Compactor,	88
	CS54 Vibratory Soil Compactor,	117	CS54 Vibratory Soil Compactor,	117	CS54 Vibratory Soil Compactor,	194	CS54 Vibratory Soil Compactor,	78	CS54 Vibratory Soil Compactor,	156
Virgin HMAC Surfacing:										
Thickness (mm)	45		45		45		45		45	
Asphalt Cement										
percent asphalt cement	5%		5%		5%		5%		5%	
Asphalt cement production energy	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul from Asphalt cement source to HMAC Plant	367 Peterbilt	1145	367 Peterbilt	1145	367 Peterbilt	1145	367 Peterbilt	1145	367 Peterbilt	1145
Asphalt Aggregate										
Aggregate production (removal from pit and crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul from HMAC aggregate Stockpile to HMAC Plant	367 Peterbilt	447	367 Peterbilt	447	367 Peterbilt	447	367 Peterbilt	447	367 Peterbilt	447
HMAC										
HMAC production energy (Plant mixing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance to HMAC Plant from site	367 Peterbilt	327	367 Peterbilt	327	367 Peterbilt	327	367 Peterbilt	327	367 Peterbilt	327
Placement	AP600D Asphalt Paver	255	AP600D Asphalt Paver	255	AP600D Asphalt Paver	255	AP600D Asphalt Paver	255	AP600D Asphalt Paver	255
Compaction	CB-534D Asphalt Compactor	91	CB-534D Asphalt Compactor	91	CB-534D Asphalt Compactor	91	CB-534D Asphalt Compactor	91	CB-534D Asphalt Compactor	91
TOTAL FUEL CONSUMPTION (L)		9049		5776		11461		11115		8929
TOTAL ENERGY CONSUMPTION (GJ)		316		202		400		388		312
		8138		4865		10319		7047		2410
		90%		84%		90%		63%		27%

	Energy Consumption (MJ)				
	Baseline	Baseline- RAP	Thickened	Drainage	Drainage - RAP
Demolition of Existing Structure					
Breaking up and removing of original structure	7,664	7,664	7,664	7,664	2,244
Waste Stucture					
haul to stockpile site	102,855	102,855	102,855	102,855	17,143
Subgrade preparation					
Production of strengthening additives	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from additive source to site					
Mixing of subgrade additives					
Shaping and compaction (hrs)					
Base					
Virgin Granular materials					
Aggregate production (removal from pit and crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from aggregate source to site	114,284	0	190,473	76,189	0
Stockpiled Recycled Granular Materials					
Recycled materials production (crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from recycled materials stockpile to site					
In-Place Recycled Granular Materials					
Milling* (May be captured in demolition above)					
Moving of recycled granular to remove subgrade from below	0	0	0	0	76,189
Drainage Layer RCP Materials					
Recycled materials production (crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from recycled materials stockpile to site	0	0	0	114,284	114,284
Modifications to in-place Granular Materials					
Production of strengthening additives					
Haul distance from additive source to site					
Mixing of base and additives					
Shaping	5,692	5,692	9,486	3,794	7,589
Compaction	6,372	6,372	10,621	4,248	8,496
Virgin HMAC Surfacing:					
Asphalt Cement					
percent asphalt cement					
Asphalt cement production energy	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from Asphalt cement source to HMAC Plant	39,999	39,999	39,999	39,999	39,999
Asphalt Aggregate					
Aggregate production (removal from pit and crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from HMAC aggregate Stockpile to HMAC Plant	15,600	15,600	15,600	15,600	15,600
HMAC					
HMAC production energy (Plant mixing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from HMAC Plant to site	11,428	11,428	11,428	11,428	11,428
Placement	8,898	8,898	8,898	8,898	8,898
Compaction	3,190	3,190	3,190	3,190	3,190
TOTAL ENERGY CONSUMPTION (MJ)	**315,982**	**201,698**	**400,214**	**388,150**	**305,060**

	Baseline	Baseline - RAP	Thickened	Drainage	Drainage - RAP
Structural Composition:					
Virgin HMAC Thickness	100 mm	100 mm	100 mm	100 mm	100 mm
Virgin Base Thickness	450 mm	300 mm	600 mm	200 mm	50 mm
Hauled-in Recycled Asphaltic Concrete Thickness	--	150 mm	--	--	150 mm
Hauled-in Recycled Portland Cement Concrete Thickness	--	--	--	250 mm	250 mm
In-Place Recycled Base	--	--			--
Total Structure Thickness	**550 mm**	**550 mm**	**700 mm**	**550 mm**	**550 mm**
Virgin HMAC	171	171	171	171	171
Virgin Base	308	206	411	137	34
Hauled-in Recycled Asphaltic Concrete	0	128	0	0	128
In-Place Recycled PCC	0	0	0	351	351
In-Place Recycled Base	0	0	0	0	0
'Total' Structure Stiffness	**479**	**504**	**582**	**659**	**683**
Virgin HMAC	1000 m³	1000 m³	1000 m³	1000 m³	1000 m³
Virgin Base	4500 m³	3000 m³	6000 m³	2000 m³	500 m³
Stockpile Crushed Recycled Asphaltic Concrete	--	1500 m³	--	--	1500 m³
Stockpile Crushed Portland Cement Concrete				2500 m³	2500 m³
Cement for Base Stabilization	--	--	--	--	--
Aggregate for HMAC	910 m³	910 m³	910 m³	910 m³	910 m³
Asphalt Oil for HMAC	50 m³	50 m³	50 m³	50 m³	50 m³
Waste Granular	5500 m³	5500 m³	5500 m³	5500 m³	5500 m³
Waste Subgrade	--	--	1500 m³	--	--
On-site Construction Equipment Resources					
Haul of Waste Granular/HMAC	15000 km	15000 km	15000 km	15000 km	0 km
Haul of Waste Subgrade	0 km	0 km	4091 km	0 km	0 km
Haul of Recycled Portland Cement Concrete	0 km	0 km	0 km	6818 km	6818 km
Haul of Recycled Asphalic Concrete	0 km	4091 km	0 km	0 km	4091 km
Haul of Virgin Base	16364 km	10909 km	21818 km	7273 km	1818 km
Haul of Aggregate to HMAC Plant	2482 km	2482 km	2482 km	2482 km	2482 km
Haul of Asphalt Oil to HMAC Plant	6364 km	6364 km	6364 km	6364 km	6364 km
Haul of HMAC to Site	1818 km	1818 km	1818 km	1818 km	1818 km
Total Truck Kilometers (Round Trip)	**42027 km**	**40664 km**	**51573 km**	**39755 km**	**23391 km**

	Baseline		Baseline		Thickened		Drainage		Drainage - RAP	
	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)	Equipment Used	Fuel (L)
Existing Road Structure										
HMAC Thickness (mm)	100		100		100		100		100	
Base Thickness (mm)	450		450		450		450		450	
Length of section for rehabilitation (m)	1000		1000		1000		1000		1000	
Width of road (m)	10		10		10		10		10	
Demolition of Existing Structure										
Breaking up and removing of original structure	M315D Wheel Excavator	447	M315D Wheel Excavator	447	M315D Wheel Excavator	447	M315D Wheel Excavator	447	RM-500 Rotary Mixer	785
Waste Stucture									950H Wheel Loader	296
HMAC & Base removal qty (m3)	5500		5500		5500		5500		5500	
Subgrade removal qty (m3)	0		0		1500		0		0	
haul to landfill	367 Peterbilt	6000	367 Peterbilt	6000	367 Peterbilt	6000	367 Peterbilt	6000	367 Peterbilt	6000
Subgrade preparation										
percent cement added (%)	0		0		0		0		0	
Production of cement materials	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from cement source to site	N/A		N/A		N/A		N/A		N/A	
Mixing of subgrade cement	N/A		N/A		N/A		N/A		N/A	
Shaping and compaction (hrs)	N/A		N/A		N/A		N/A		N/A	
Base										
Virgin Granular materials										
Quantity (m3)	4500		3000		6000		2000		500	
Aggregate production (removal from pit and crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from aggregate source to site	367 Peterbilt	6545	367 Peterbilt	4364	367 Peterbilt	8727	367 Peterbilt	2909	N/A	
Stockpiled Recycled Granular Materials										
Quantity (m3)	0		1500		0		0		1500	
Recycled materials production (crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from recycled materials stockpile to site	N/A		N/A		N/A		N/A		N/A	
In-Place Recycled Granular Materials										
Soil Quantity (m3)	0		0		0		0		0	
Milling* (May be captured category in demolition above)	N/A		N/A		N/A		Accounted Above		Accounted Above	
Moving of recycled granular to remove subgrade from below	N/A		N/A		N/A		N/A		367 Peterbilt	0
Drainage Layer RCP Materials										
Soil Quantity (m3)	0		0		0		2500		2500	
Recycled materials production (crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from recycled materials stockpile to site	N/A		N/A		N/A		367 Peterbilt	3636	367 Peterbilt	3636
Modifications to in-place Granular Materials										
Soil Quantity (m3)	0		0		0		0		1500	
Percent cement	N/A		N/A		N/A		0%		0%	
Cement Quantity (m3)	0		0		0		0		0	
Production of cement materials	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
Haul distance from cement source to site	N/A		N/A		N/A		N/A		N/A	0
Percent emulsion	N/A		N/A		N/A		N/A		N/A	
Production of emulsion material	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance from emulsion source to site	N/A		N/A		N/A		N/A		N/A	
Mixing of base & cement and/or emulsion	N/A		N/A		N/A		RM-500 Rotary Mixer	0	RM-500 Rotary Mixer	169
Shaping and compaction	12M Motor Grader,	326	12M Motor Grader,	326	12M Motor Grader,	435	12M Motor Grader,	145	12M Motor Grader,	254
	CP76 Vibratory Soil Compactor,	132	CP76 Vibratory Soil Compactor,	132	CP76 Vibratory Soil Compactor,	176	CP76 Vibratory Soil Compactor,	59	CP76 Vibratory Soil Compactor,	102
	CS54 Vibratory Soil Compactor,	233	CS54 Vibratory Soil Compactor,	233	CS54 Vibratory Soil Compactor,	311	CS54 Vibratory Soil Compactor,	104	CS54 Vibratory Soil Compactor,	181
Virgin HMAC Surfacing:										
Thickness (mm)	100		100		100		100		100	
Asphalt Cement										
percent asphalt cement	5%		5%		5%		5%		5%	
Asphalt cement production energy	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul from Asphalt cement source to HMAC Plant	367 Peterbilt	2545	367 Peterbilt	2545	367 Peterbilt	2545	367 Peterbilt	2545	367 Peterbilt	2545
Asphalt Aggregate										
Aggregate production (removal from pit and crushing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul from HMAC aggregate Stockpile to HMAC Plant	367 Peterbilt	993	367 Peterbilt	993	367 Peterbilt	993	367 Peterbilt	993	367 Peterbilt	993
HMAC										
HMAC production energy (Plant mixing)	Out of Scope		Out of Scope		Out of Scope		Out of Scope		Out of Scope	
haul distance to HMAC Plant from site	367 Peterbilt	727	367 Peterbilt	727	367 Peterbilt	727	367 Peterbilt	727	367 Peterbilt	727
Placement	AP600D Asphalt Paver	566	AP600D Asphalt Paver	566	AP600D Asphalt Paver	566	AP600D Asphalt Paver	566	AP600D Asphalt Paver	566
Compaction	CB-534D Asphalt Compactor	203	CB-534D Asphalt Compactor	203	CB-534D Asphalt Compactor	203	CB-534D Asphalt Compactor	203	CB-534D Asphalt Compactor	203
TOTAL FUEL CONSUMPTION (L)		18718		16536		21130		18334		16459
TOTAL ENERGY CONSUMPTION (GJ)		**654**		**577**		**738**		**640**		**575**

	Energy Consumption (MJ)				
	Baseline	**Baseline - RAP**	**Thickened**	**Drainage**	**Drainage - RAP**
Demolition of Existing Structure					
Breaking up and removing of original structure	15,611	15,611	15,611	15,611	27,428
Waste Stucture					
haul to stockpile site	209,520	209,520	209,520	209,520	209,520
Subgrade preparation					
Production of strengthening additives	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from additive source to site					
Mixing of subgrade additives					
Shaping and compaction (hrs)					
Base					
Virgin Granular materials					
Aggregate production (removal from pit and crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from aggregate source to site	228,567	152,378	304,756	101,585	0
Stockpiled Recycled Granular Materials					
Recycled materials production (crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from recycled materials stockpile to site					
In-Place Recycled Granular Materials					
Milling* (May be captured in demolition above)					
Moving of recycled granular to remove subgrade from below	0	0	0	0	0
Drainage Layer RCP Materials					
Recycled materials production (crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul distance from recycled materials stockpile to site	0	0	0	126,982	126,982
Modifications to in-place Granular Materials					
Production of strengthening additives					
Haul distance from additive source to site					
Mixing of base and additives					
Shaping	11,383	11,383	15,178	5,059	8,854
Compaction	12,745	12,745	16,993	5,664	9,912
Virgin HMAC Surfacing:					
Asphalt Cement					
percent asphalt cement					
Asphalt cement production energy	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from Asphalt cement source to HMAC Plant	88,887	88,887	88,887	88,887	88,887
Asphalt Aggregate					
Aggregate production (removal from pit and crushing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from HMAC aggregate Stockpile to HMAC Plant	34,666	34,666	34,666	34,666	34,666
HMAC					
HMAC production energy (Plant mixing)	Out of Scope	Out of Scope	Out of Scope	Out of Scope	Out of Scope
haul from HMAC Plant to site	25,396	25,396	25,396	25,396	25,396
Placement	19,773	19,773	19,773	19,773	19,773
Compaction	7,088	7,088	7,088	7,088	7,088
TOTAL ENERGY CONSUMPTION (MJ)	**653,638**	**577,449**	**737,870**	**640,234**	**558,507**

APPENDIX F
ROADWAY MODELLING RESULTS

LOCAL ROADS

1) Subgrade Sensitivtiy (Subgrade Types, Varied Moisture Content)

Case 1A
45 mm HMAC
225 mm Good Base (Dry)
∞ Good SM-SC (Opt - 3% Dry)

Case 1C
45 mm HMAC
225 mm Good Base (Dry)
∞ Poor CH (Opt - 3% Dry)

Case 1F
45 mm HMAC
225 mm Good Base (Wet)
∞ Good SM-SC (Opt + 3% Wet)

Case 1H
45 mm HMAC
225 mm Good Base (Wet)
∞ Poor CH (Opt + 3% Wet)

2) Base Course Material Sensitivity (Thickness)

Case 2H
45 mm HMAC
375 mm Good Base (Wet)
∞ Poor CH (Opt + 3% Wet)

3) Base Course Material Sensitivity (RAP, Good and Poor base)

Case 3A
45 mm HMAC
225 mm RAP Base
∞ Good SM-SC (Opt - 3% Dry)

Case 3B
45 mm HMAC
225 mm RAP Base
∞ Poor CH (Opt + 3% Wet)

Case 3C
45 mm HMAC
225 mm Poor Base (Dry)
∞ Good SM-SC (Opt - 3% Dry)

4) Effect of Drainage/Strain Dissapation Layer

Case 4D
45 mm HMAC
150 mm Good Base (Dry)
225 mm PCC Drainage Rock
∞ Poor CH (Opt + 3% Wet)

Local Roadways

Case 1A

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	97.468	125.654	61.679	-147.623	-16.032	-350.62	575.506
BOTTOM OF HMAC	140.15	57.187	310.428	-136.278	-351.882	-112.521	559.997
TOP OF SUBGRADE	105.717	3.504	226.122	-76.883	-492.97	-55.341	445.037

Max Layer Strains

Layer	Strain $_{XX}$ Tensile	Strain $_{XX}$ Compressive	Strain $_{YY}$ Tensile	Strain $_{YY}$ Compressive	Strain $_{ZZ}$ Tensile	Strain $_{ZZ}$ Compressive	Shear Strain
1	140	-148	126	-352	310	-351	615
2	159	-123	28	-1106	358	-116	1108
3	106	-77	4	-493	226	-55	445
4	29	-19	1	-102	44	-12	85
5	6	-5	2	-29	12	-8	24
6	5	-5	1	-18	3	-3	10
7	5	-5	2	-11	1	-1	3
8	5	0	0	-2	0	-1	1

Case 1C

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	95.791	189.975	31.834	-180.274	-5.751	-413.239	596.269
BOTTOM OF HMAC	129.21	80.999	299.15	-151.364	-328.66	-127.252	582.899
TOP OF SUBGRADE	183.361	0.868	381.267	-139.164	-923.849	-81.628	741.849

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	129	-180	190	-329	299	-413	637
2	181	-134	30	-1082	388	-124	1209
3	183	-139	1	-924	381	-82	742
4	44	-31	1	-202	72	-22	152
5	10	-10	1	-74	20	-15	45
6	10	-11	1	-45	6	-6	21
7	11	-11	2	-29	1	-1	7
8	21	0	0	-4	2	-2	3

Case 1F

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	304.286	784.831	225.652	-482.338	-94.326	-1188.361	1456.742
BOTTOM OF HMAC	548.295	81.978	1295.179	-318.721	-1129.088	-224.991	1435.132
TOP OF SUBGRADE	1104.157	196.795	1790.098	-831.668	-3965.641	-447.648	3335.182

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	548	-482	785	-1129	1295	-1188	1556
2	1181	-855	462	-3253	1890	-493	4309
3	1104	-832	197	-3966	1790	-448	3335
4	275	-186	34	-858	409	-116	730
5	38	-26	6	-174	72	-46	141
6	19	-20	4	-79	17	-16	46
7	17	-18	5	-41	2	-2	12
8	0	-24	17	0	3	-3	26

Case 1H

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	367.143	938.729	240.958	-561.206	-126.389	-1368.732	1589.089
BOTTOM OF HMAC	623.902	77.996	1440.315	-331.339	-1252.015	-223.926	1570.417
TOP OF SUBGRADE	1858.006	470.217	2613.244	-1394.941	-4617.291	-616.204	4392.29

Max Layer Strains

Layer	Strain $_{xx}$		Strain $_{YY}$		Strain $_{zz}$		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	624	-561	939	-1252	1440	-1369	1697
2	1788	-1342	492	-4227	2564	-600	5017
3	1858	-1395	470	-4617	2613	-616	4392
4	595	-340	257	-1379	916	-321	1332
5	100	-66	55	-245	181	-107	297
6	52	-55	51	-99	46	-41	102
7	50	-52	49	-61	7	-8	30
8	0	-61	62	0	8	-7	65

Case 2H

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	344.436	870.986	220.844	-528.341	-109.983	-1286.693	1525.769
BOTTOM OF HMAC	594.294	70.851	1369.108	-314.272	-1196.711	-216.356	1505.596
TOP OF SUBGRADE	1546.535	301.943	2099.555	-1119.413	-3633.832	-480.054	3186.778

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	594	-528	871	-1197	1369	-1287	1629
2	1561	-1132	338	-3627	2121	-483	4806
3	1547	-1119	302	-3634	2100	-480	3187
4	573	-322	252	-1335	889	-313	1289
5	97	-65	53	-240	176	-105	289
6	52	-55	51	-98	45	-41	102
7	50	-52	49	-61	7	-8	30
8	0	-83	68	0	13	-11	56

Case 3A

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	47.215	85.083	19.235	-86.731	-44.406	-204.521	377.183
BOTTOM OF HMAC	57.922	66.275	112.041	-100.545	-204.992	-70.614	353.538
TOP OF SUBGRADE	89.275	3.835	184.325	-63.714	-394.048	-37.586	332.816

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	58	-101	85	-205	112	-205	388
2	86	-88	22	-451	183	-68	544
3	89	-64	4	-394	184	-38	333
4	24	-15	1	-90	38	-12	74
5	6	-5	2	-27	11	-8	22
6	5	-5	1	-18	3	-3	10
7	5	-5	2	-11	0	0	3
8	4	0	0	-2	0	0	1

Case 3B

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	118.264	298.194	62.002	-192.868	-52.303	-422.289	436.301
BOTTOM OF HMAC	75.783	163.648	43.903	-156.528	-127.574	-163.314	423.931
TOP OF SUBGRADE	340.949	127.307	624.631	-191.656	-1001.813	-168.107	749.264

Max Layer Strains

Layer	Strain $_{XX}$		Strain $_{YY}$		Strain $_{ZZ}$		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	118	-193	298	-128	62	-422	458
2	297	-172	32	-510	569	-115	749
3	346	-192	247	-1002	625	-288	830
4	180	-119	220	-517	391	-274	549
5	51	-58	52	-150	104	-87	191
6	52	-56	52	-86	32	-31	90
7	51	-54	51	-61	6	-6	30
8	31	-58	49	-24	7	-6	17

Case 3C

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	93.282	86.033	74.928	-129.685	-28.25	-310.568	580.046
BOTTOM OF HMAC	165.495	43.098	346.357	-122.767	-397.714	-98.476	565.992
TOP OF SUBGRADE	137.025	4.217	272.831	-95.186	-574.136	-62.386	511.916

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	165	-130	86	-398	346	-311	618
2	222	-138	59	-790	400	-99	1107
3	137	-95	4	-574	273	-62	512
4	31	-21	1	-108	48	-12	90
5	6	-5	2	-29	12	-8	24
6	5	-5	1	-18	3	-3	10
7	5	-5	1	-11	0	0	3
8	2	-1	1	-1	0	0	1

Case 4D

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	81.909	151.142	66.627	-158.208	-49.531	-371.601	547.728
BOTTOM OF HMAC	130.813	117.364	241.941	-166.248	-293.766	-176.07	526.044
TOP OF SUBGRADE	134.358	57.112	238.073	-62.492	-408.912	-90.527	263.563

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	131	-166	151	-294	242	-372	581
2	145	-163	102	-1109	281	-183	979
3	113	-56	16	-129	206	-58	182
4	138	-92	92	-413	244	-137	363
5	103	-92	87	-299	199	-137	326
6	51	-55	51	-106	58	-49	127
7	53	-55	53	-75	19	-18	79
8	52	-54	51	-60	3	-4	29

ARTERIAL ROADS

1) Subgrade Sensitivtiy (Subgrade Types, Varied Moisture Content)

Case 1A

100 mm HMAC
150 mm Good Base (Dry)
300 mm Subbase (Dry)
∞ Good SM-SC (Opt - 3% Dry)

Case 1C

100 mm HMAC
150 mm Good Base (Dry)
300 mm Subbase (Dry)
∞ Poor CH (Opt - 3% Dry)

Case 1F

100 mm HMAC
150 mm Good Base (Wet)
300 mm Subbase (Wet)
∞ Good SM-SC (Opt + 3% Wet)

Case 1G

100 mm HMAC
150 mm Good Base (Opt)
300 mm Subbase (Wet)
∞ Poor CH (Opt + 3% Wet)

Case 1H

100 mm HMAC
150 mm Good Base (Wet)
300 mm Subbase (Wet)
∞ Poor CH (Opt + 3% Wet)

2) Base Course Material Sensitivity (Thickness)

Case 2H

100 mm HMAC
300 mm Good Base (Wet)
300 mm Subbase (Wet)
∞ Poor CH (Opt + 3% Wet)

3) Base Course Material Sensitivity (RAP, Good and Poor base)

Case 3A

100 mm HMAC
150 mm RAP Base
300 mm Subbase (Dry)
∞ Good SM-SC (Opt - 3% Dry)

Case 3B

100 mm HMAC
150 mm RAP Base
300 mm Subbase (Wet)
∞ Poor CH (Opt + 3% Wet)

Case 3C

100 mm	HMAC
150 mm	Poor Base (Dry)
300 mm	Subbase (Dry)
∞	Good SM-SC (Opt - 3% Dry)

4) Effect of Drainage/Strain Dissipation Layer

Case 4D.1

100 mm	HMAC
200 mm	Good Base (Dry)
250 mm	PCC Drainage Rock
∞	Poor CH (Opt + 3% Wet)

Case 4D.2

100 mm	HMAC
200 mm	RAP Base
250 mm	PCC Drainage Rock
∞	Poor CH (Opt + 3% Wet)

Arterial Modeling

Case 1A

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainYZ (T)	StrainXZ (T)	StrainXY (T)	Shear Strain
TOP OF HMAC	-57.402	-97.391	-41.896	-340.485	-106.695	-277.294	340.485
BOTTOM OF HMAC	-107.547	-21.722	-250.68	-314.182	-138.409	-233.492	314.182
TOP OF SUBGRADE	-54.803	-2.223	-81.521	-162.352	-42.899	-147.614	162.352

Max Layer Strains

Layer	Strain $_{XX}$		Strain $_{YY}$		Strain $_{ZZ}$		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	108	-93	97	-269	251	-212	400
2	122	-77	11	-645	274	-60	585
3	100	-74	6	-459	204	-50	394
4	55	-43	2	-205	82	-21	162
5	27	-18	1	-98	42	-12	82
6	6	-5	2	-28	12	-8	23
7	5	-5	1	-18	3	-3	10
8	5	-5	2	-11	0	0	3

Case 1C

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	54.263	117.725	33.902	-105.358	-35.016	-231.798	343.77
BOTTOM OF HMAC	104.425	27.3	249.698	-72.686	-264.006	-59.726	321.628
TOP OF SUBGRADE	89.095	0.634	140.354	-67.366	-386.239	-31.8	280.674

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	104	-105	118	-264	250	-232	408
2	121	-82	13	-636	275	-60	607
3	102	-77	9	-435	210	-44	432
4	89	-67	1	-386	140	-32	281
5	44	-27	1	-190	72	-24	144
6	9	-10	1	-71	19	-14	44
7	10	-11	0	-43	5	-5	21
8	10	-11	1	-28	1	-1	6

Case 1F

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	224.313	436.602	128.336	-300.135	-75.232	-610.192	546.456
BOTTOM OF HMAC	351.399	66.199	706.651	-233.117	-638.828	-126.628	579.46
TOP OF SUBGRADE	460.279	98.764	631.362	-337.796	-1470.953	-188.86	1165.032

Max Layer Strains

Layer	Strain XX Tensile	Strain XX Compressive	Strain YY Tensile	Strain YY Compressive	Strain ZZ Tensile	Strain ZZ Compressive	Shear Strain
1	351	-300	0	0	Tensile(+)	Compressive(-)	0
2	719	-559	437	-639	0	0	0
3	769	-589	182	-1739	707	-610	0
4	460	-338	222	-1898	992	-264	760
5	205	-125	99	-1471	1015	-287	1590
6	34	-27	53	-702	631	-189	1605
7	19	-21	6	-170	323	-116	1443
8	17	-19	4	-80	71	-50	1522

Case 1H

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	243.98	480.581	122.489	-327.936	-73.829	-660.07	575.548
BOTTOM OF HMAC	380.854	52.188	750.233	-237.709	-675.681	-120.628	609.023
TOP OF SUBGRADE	678.787	283.788	1018.147	-424.352	-1641.455	-334.059	1518.707

Max Layer Strains

Layer	Strain $_{XX}$		Strain $_{YY}$		Strain $_{ZZ}$		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	381	-328	481	-676	750	-660	793
2	713	-503	140	-1738	1034	-233	1823
3	765	-519	269	-1894	1075	-332	1818
4	679	-424	286	-1641	1018	-334	1684
5	378	-176	267	-942	659	-323	1663
6	77	-63	55	-205	151	-106	1400
7	52	-56	52	-96	42	-39	838
8	51	-53	50	-62	7	-7	234

Case 2H

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	233.32	462.063	118.309	-315.001	-69.975	-641.095	563.357
BOTTOM OF HMAC	368.156	51.646	731.499	-227.233	-659.001	-118.309	596.593
TOP OF SUBGRADE	478.561	271.274	775.044	-250.451	-1193.887	-316.301	1639.905

Max Layer Strains

Layer	Strain $_{XX}$ Tensile	Strain $_{XX}$ Compressive	Strain $_{YY}$ Tensile	Strain $_{YY}$ Compressive	Strain $_{ZZ}$ Tensile	Strain $_{ZZ}$ Compressive	Shear Strain
1	368	-315	462	-659	731	-641	779
2	727	-489	191	-1754	1020	-266	1792
3	722	-481	265	-1738	1018	-314	1542
4	479	-250	271	-1194	775	-316	1672
5	365	-170	256	-914	635	-311	1380
6	76	-62	52	-203	148	-103	1485
7	52	-56	52	-95	42	-39	816
8	51	-53	50	-62	7	-7	231

Case 3A

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	39.052	75.344	22.479	-69.189	-92.487	-148.487	295.196
BOTTOM OF HMAC	59.362	18.856	135.861	-43.048	-189.715	-34.773	273.13
TOP OF SUBGRADE	47.185	2.824	71.125	-35.211	-179.474	-17.759	139.818

Max Layer Strains

Layer	Strain $_{XX}$ Tensile	Strain $_{XX}$ Compressive	Strain $_{YY}$ Tensile	Strain $_{YY}$ Compressive	Strain $_{ZZ}$ Tensile	Strain $_{ZZ}$ Compressive	Shear Strain
1	59	-69	75	-190	136	-148	309
2	90	-62	9	-303	187	-39	375
3	94	-67	5	-400	189	-40	335
4	47	-35	3	-179	71	-18	343
5	24	-15	1	-89	38	-12	313
6	6	-5	2	-27	11	-8	128
7	5	-5	1	-18	3	-3	67
8	5	-5	1	-11	0	0	22

Case 3B

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	106.349	233.306	54.218	-158.156	-42.916	-323.694	332.986
BOTTOM OF HMAC	74.538	16.092	159.131	-46.181	-192.993	-48.984	367.698
TOP OF SUBGRADE	279.161	245.994	523.272	-121.24	-774.81	-285.761	721.953

Max Layer Strains

	Strain $_{XX}$		Strain $_{YY}$		Strain $_{ZZ}$		Shear Strain
Layer	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	106	-158	233	-193	159	-324	400
2	271	-159	37	-389	509	-99	507
3	304	-176	230	-869	556	-272	740
4	279	-121	246	-775	523	-286	798
5	176	-117	220	-507	386	-272	582
6	51	-58	52	-148	102	-86	607
7	52	-56	53	-86	31	-30	455
8	52	-54	51	-61	5	-5	169

Case 3C

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	60.431	88.578	50.785	-90.041	-60.295	-208.828	345.835
BOTTOM OF HMAC	123.253	16.842	273.369	-72.746	-291.494	-54.509	318.619
TOP OF SUBGRADE	72.464	3.699	105.669	-56.853	-250.936	-26.534	199.754

Max Layer Strains

Layer	Strain XX		Strain YY		Strain ZZ		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	123	-90	89	-291	273	-209	406
2	145	-102	44	-510	303	-66	597
3	143	-106	42	-438	260	-64	443
4	72	-57	4	-251	106	-27	540
5	34	-23	1	-113	51	-14	414
6	7	-5	2	-30	13	-9	189
7	5	-5	1	-18	4	-3	91
8	5	-5	1	-11	1	-1	24

Case 4D1

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	42.82	94.302	37.132	-92.966	-46.889	-212.171	331.307
BOTTOM OF HMAC	96.443	43.819	213.679	-72.769	-237.441	-78.357	294.03
TOP OF SUBGRADE	85.387	54.131	140.687	-50.233	-268.024	-62.299	170.114

Max Layer Strains

Layer	Strain XX		Strain YY		Strain ZZ		Shear Strain
	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	96	-93	94	-237	214	-212	377
2	104	-76	34	-619	227	-81	531
3	77	-30	9	-83	124	-42	312
4	85	-68	74	-268	141	-75	560
5	76	-77	75	-218	127	-82	187
6	50	-53	50	-87	40	-33	111
7	52	-55	52	-69	14	-13	266
8	52	-54	51	-59	3	-3	108

Case 4D2

Conventional Strains

Layer	StrainXX (T)	StrainYY (T)	StrainZZ (T)	StrainXX (C)	StrainYY (C)	StrainZZ(C)	Shear Strain
TOP OF HMAC	19.731	68.31	36.849	-62.278	-98.631	-131.601	282.438
BOTTOM OF HMAC	52.527	43.708	105.896	-49.956	-168.153	-62.639	237.871
TOP OF SUBGRADE	67.387	56.37	115.462	-50.565	-223.2	-56.115	136.246

Max Layer Strains

	Strain $_{XX}$		Strain $_{YY}$		Strain $_{ZZ}$		Shear Strain
Layer	Tensile	Compressive	Tensile	Compressive	Tensile	Compressive	
1	53	-66	68	-168	106	-132	282
2	55	-46	23	-278	111	-55	311
3	64	-19	9	-72	107	-42	255
4	67	-68	77	-223	115	-64	306
5	60	-76	77	-177	98	-67	144
6	50	-53	50	-77	30	-26	86
7	53	-54	52	-66	10	-10	241
8	52	-54	51	-59	2	-2	98